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Slope Stabilization Using Recycled Plastic Pins:

Phase II - Assessment in Varied Site Conditions

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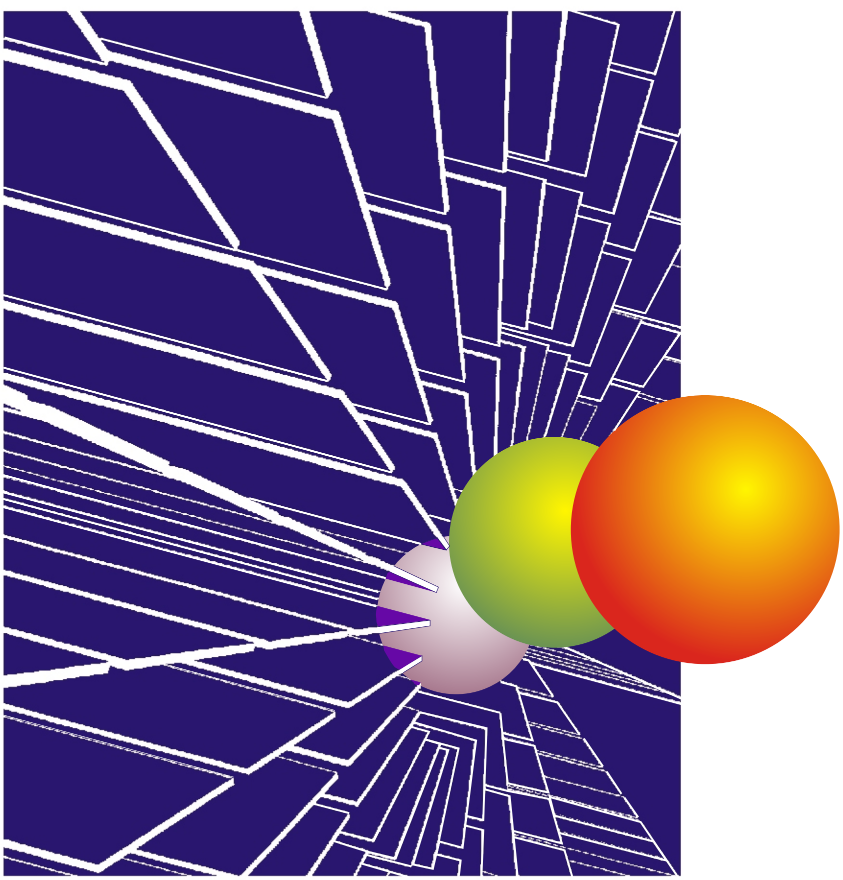

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Slope Stabilization Using Recycled Plastic Pins: Phase II – Assessment in Varied Site Conditions

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16. Abstract

A new technique for stabilizing surficial slope failures using recycled plastic reinforcing members has been developed. Preliminary evaluations performed in Phase I of the project indicated that the technique could be constructed cost effectively and provide sufficient resistance to stabilize surficial slides, at least in the short-term. The evaluation was therefore expanded to include test sections at five different sites with varied conditions. This report documents the activities undertaken to establish the additional test sites and the results of performance monitoring of each of these sites since installation.

The test sites established during Phase II include two sites in southern Kansas City Missouri, one site near Stewartsville Missouri, one site near Emma Missouri, and one site near Fulton Missouri. The slopes at these sites include both embankment and cut slopes with heights ranging from 15- to 46-ft (5- to 14-m) and slope inclinations from 2.2:1 (H:V) to 3.2:1. Subsurface conditions for the different slopes also vary as do the stabilization schemes selected for installation at each site.

Based on results of performance monitoring at each of the test sites to date, the following conclusions are drawn: (1) surficial slides can be effectively stabilized using recycled plastic reinforcing members placed in a 3-ft by 3-ft (0.9-m by 0.9-m) staggered arrangement over the entire slide area; (2) surficial slides may possibly be stabilized using more widely spaced reinforcing members, with substantial cost savings, but additional monitoring of the field sites is needed to more definitively establish minimum required reinforcement patterns; (3) the response of the demonstration sites has followed a consistent, three stage pattern that includes a period of little movement and little load transfer to the reinforcement, a period of increased movement and increased load transfer in response to increased precipitation, and finally, a period where movements and loads are observed to stabilize as a result of reaching an equilibrium state; and, (4) the efficiency of the installation improved dramatically with slight modifications to the installation equipment and installation technique.

Given the cost effectiveness and successful demonstration of the technique to date, it is recommended that the technique be implemented in “production” operations on a trial basis. Simultaneously, it is recommended that monitoring of the test sites established during the project be continued for a period of one year to better establish the performance of test sections stabilized with different reinforcement patterns. The data acquired during this period can then be used to calibrate the current design method. The calibrated method can then, in turn, be used to develop a series of practical design tools such as design charts, tables, or “rules-of-thumb” to further facilitate widespread implementation of the technique.

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Executive Summary

The objective of the project entitled "Slope Stabilization Using Recycled Plastic Pins" is to develop, evaluate, and document a technique for stabilization of surficial slope failures using recycled plastic reinforcing members (RPP). The project is being undertaken in three sequential phases to permit project accomplishments to be evaluated at logical intervals and the scope of work to be refined based on results of activities undertaken throughout the project. Phase I of the project, which was initiated in January 1999 and completed in June 2000, served as a “proof of concept” phase wherein a single slope was stabilized using recycled plastic members. Activities undertaken during the first phase are documented in the Phase I final report (Loehr et al, 2000). Phase II of the project was initiated in October 2000 to expand the evaluation of the technique. The general objectives of Phase II have been to stabilize slopes in a variety of soil-site conditions using varying reinforcement schemes, to commence and continue performance monitoring of the stabilized sites, and to expand the economic evaluation of the RPP technique. This report describes the activities performed during Phase II of the project and serves as the final report for Phase II.

Test sections were established at five sites during Phase II. Well over 50 candidate test sites were evaluated prior to making the final site selections. Selected sites were chosen to provide for evaluation of the stabilization technique in a variety of different conditions (e.g. slope type, slope height, slope inclination, water conditions, etc.) while at the same time providing opportunity to evaluate alternative stabilization schemes. Two of the selected sites were located in District 4 on Interstate 435 in southern Kansas City. Additional sites were located in District 1 on U.S. Highway 36 near Stewartsville Missouri, in District 2 on Interstate 70 near Emma Missouri, and in District 5 on U.S. Highway 54 near Fulton Missouri. Two of the slopes are excavated slopes; the remaining slopes are embankment slopes.

At each of the selected sites, extensive site investigations and laboratory testing programs were performed to establish the conditions that are believed to have led to previous failures at the sites. Stability analyses were then performed to evaluate the stability of the respective slopes for different potential stabilization schemes. Results of these analyses were then used to select the stabilization scheme(s) to be used at the respective sites. Variable stabilization schemes were used at different sites, and within single sites, so that the technique could be optimized based on the costs and performance of each of the stabilized sections. Selected stabilization schemes varied from schemes with relatively closely spaced members that were believed to very likely stabilize the test slopes to schemes with relatively widely spaced members that are not likely to provide long-term stabilization.

The selected stabilization measures were then installed at each site. Installation at the I435-Kansas City sites was completed in December 2001. Installation at the US36-Stewartsville site was completed in May 2002. Installations at the I70-Emma and US54-Fulton sites were completed in January 2003. Following installation, a suite of field instrumentation was installed at each of the sites to permit the performance of the respective stabilization measures to be monitored. Instrumentation included slope inclinometers, standpipe piezometers and other soil moisture and pore pressures sensors, and several instrumented reinforcing members to monitor the loads being carried by the members. Field instrumentation at each site has been periodically monitored since installation and the data

has been analyzed and interpreted to establish both a qualitative and quantitative interpretation of the performance of each test section.

Results obtained from monitoring the performance of the respective sites have allowed a number of important conclusions to be drawn. The most significant of these conclusions include:

(1) Surficial slope failures can be effectively stabilized by installing recycled plastic members across the entire slide area in a 3-ft by 3-ft (0.9-m by 0.9-m) staggered arrangement.
(2) Surficial slope failures may possibly also be effectively stabilized using more economical arrangements of reinforcing members. However, additional monitoring is needed to confirm or refute this possibility.
(3) A consistent pattern of behavior has been observed at each of the stabilized sites, which has facilitated the current understanding of load transfer mechanisms for this type of stabilization and may lead to tangible recommendations regarding the most effective methods for applying the technique in the future. This pattern of behavior could not have been identified without the information provided by the field instrumentation.
(4) The efficiency of the installation process has improved significantly over that achieved during Phase I.
(5) Unit costs per installed member for stabilization of the test sites have remained relatively constant over the duration of the project. However, unit costs per unit area of slope stabilized have varied substantially depending on the particular scheme(s) used at each site. Nominal unit costs for the stabilization technique appear to range from approximately $4.50/ft^2 ($48/m^2) for schemes with relatively closely spaced members to less than $1.00/ft^2 ($11/m^2) for schemes with widely spaced members. However, it remains to be determined whether the less costly stabilization schemes provide effective long-term stabilization.

Based on these conclusions and others developed during the project, it is recommended that additional sites begin to be stabilized in "production" operations on a trial basis. Use of the technique in normal operations will help to identify potential issues to be addressed for more widespread implementation while at the same time making use of a new, cost-effective stabilization technique. At the same time, three additional general tasks are recommended to be accomplished before widespread implementation can occur. These general tasks include:

(1) Correlating field performance with the theoretical stability predicted by the design method developed during Phase I so that decision makers can make effective decisions regarding the trade-offs between cost and expected performance;
(2) Calibrating the design method based on these results and developing simple design tools such as charts, tables, or even "rules-of-thumb"; and
(3) Developing appropriate technology transfer materials to ensure that the technique is applied in appropriate conditions and designed and constructed properly.

Accomplishing these tasks will require continued monitoring of the established test sites as well as additional analysis of the performance of these sites. More detailed recommendations regarding each of these general tasks, including more specific sub-tasks, are provided in Chapter 10. Completion of these tasks can be accomplished in Phase III of the project, after which the method can be reliably and effectively implemented on a widespread basis.

Acknowledgements

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Chapter 1. Introduction

The objective of the project entitled "Slope Stabilization Using Recycled Plastic Pins" is to develop, evaluate, and document a technique for stabilization of surficial slope failures using recycled plastic reinforcing members (RPP). The project is being undertaken in three sequential phases described in more detail below to provide for logical evaluation of project accomplishments and refinement of the scope of work based on results of activities undertaken throughout the project. This report describes the activities performed during Phase II of the project and serves as the final report for Phase II.

1.1. Motivation

Slope failures and landslides constitute significant hazards to all types of both public and private infrastructure. Total direct costs for maintenance and repair of landslides involving major U.S. highways alone (roughly 20 percent of all U.S. highways and roads) were recently estimated to exceed $100 million annually (TRB, 1996). In the same study, indirect costs attributed to loss of revenue, use, or access to facilities as a result of landslides were conservatively estimated to equal or exceed direct costs. Costs for maintaining slopes for other highways, roads, levees, and railroads maintained by government and private agencies such as county and city governments, the U.S. Forest Service, the U.S. Army Corps of Engineers, the National Parks Service, and the railroad industry significantly increase the total costs for landslide repairs.

A significant, but largely neglected, toll of landslides is the costs associated with routine maintenance and repair of "surficial" slope failures. Costs for repair of such slides were not explicitly included in the above referenced study because of limited record keeping for these types of slides by most state departments of transportation. However, the authors of the TRB study conservatively estimated that costs for repair of minor slides equal or exceed costs associated with repair of major landslides. This estimate is supported by the Missouri Department of Transportation's (MoDOT) experience with surficial slide problems, which are estimated to cost on the order of $1 million per year on average. Many other state departments of transportation have similar problems with similarly high, or even higher annual costs. All available evidence clearly indicates that the cumulative costs for repair of many surficial slides can become extremely large, despite the fact that costs for repair of individual slides are generally low. In addition, minor failures often constitute significant hazards to infrastructure users (e.g. from damage to guard rails, shoulders, or portions of road surface) and, if not properly maintained, often progress into more serious problems requiring more extensive and costly repairs.

The premise of the project is that slender structural members manufactured from recycled plastics can be used to effectively reinforce slopes as illustrated in Figure 1.1. As shown in the figure, recycled plastic reinforcing members are installed in the slope to intercept potential sliding surfaces and provide the resistance needed to maintain the long-term stability of the slope. Using recycled plastic members for stabilization has several potential advantages over more common civil engineering materials. Plastic members are less susceptible to degradation by chemical and biological attack than other structural materials and are lightweight, meaning smaller installation equipment and lower transport costs. Plastic members also present less of an obstruction if future construction (e.g.

underground utilities) must traverse a stabilized site. Using recycled plastics also has environmental and political benefits as it reduces the volume of waste entering landfills and provides additional markets for recycled plastic. Development of a cost effective means for using these materials while providing long-term stabilization therefore clearly has numerous advantages for agencies like MoDOT.

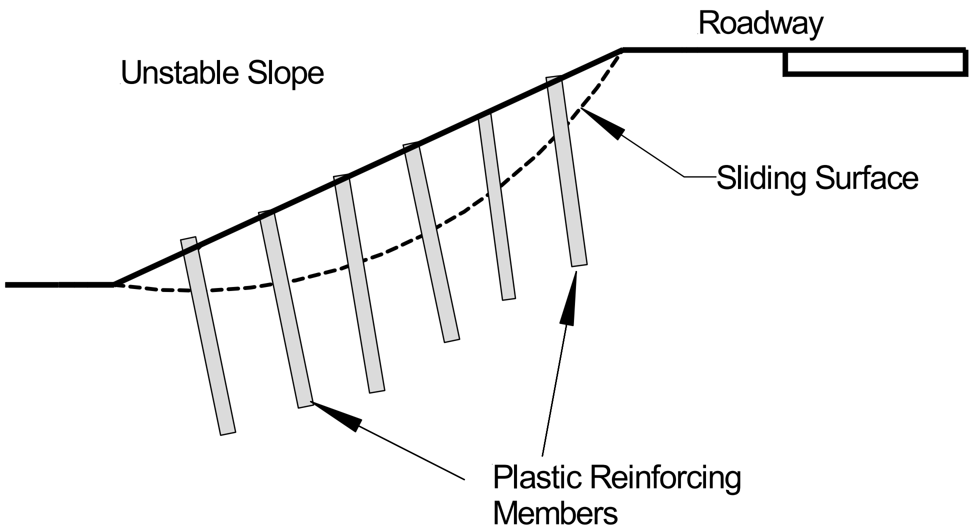


Figure 1.1 Stabilization of surficial slope failures with recycled plastic reinforcement.

1.2. Background

Because no previous attempts to utilize recycled plastic members in similar applications had been undertaken, the project was developed to be performed in three phases. Phase I of the project, which was initiated in January 1999, was intended to serve as a "proof of concept" phase, wherein a single slope was stabilized with recycled plastic members. The slope selected for stabilization in Phase I, located on Interstate 70 near Emma Missouri, was successfully stabilized in November 1999. Additional activities undertaken during Phase I included basic characterization of the engineering properties of recycled plastic members, evaluation of the long-term stability of recycled plastics when subjected to potentially detrimental environmental conditions, and installation of instrumentation for monitoring the performance of the stabilized slope. All of these activities are summarized in the final report for Phase I (Loehr et al, 2000).

Following completion of Phase I in June 2000, four broad issues remained to be addressed:

- Determining the range of applicability for using recycled plastic members for slope stabilization (e.g. soil type, slope geometry, etc.),
- Validating the assumptions inherent in the design methodology and optimizing placement of reinforcing members,
- Establishing the economics of stabilization with slender reinforcement as compared to other current and potential stabilization measures, and

- Developing and documenting formal procedures for design and construction of slope stabilization measures and technology transfer activities.

Phase II of the project was initiated in October 2000 to expand the evaluation and demonstration of the technique to begin addressing these issues by establishing additional test sites in differing site conditions, initiating additional performance monitoring to determine the load transfer mechanisms for recycled plastic reinforcement, and acquiring additional cost data for the technique. Phase III is expected to involve final analysis of the performance of the stabilized sites, calibration and final development of a suitable design procedure, development of design and construction guidelines, and technology transfer activities.

1.3. Project Objectives and Tasks

The general objectives of Phase II have been to stabilize slopes in a variety of soil-site conditions using varying reinforcement schemes, to commence and continue performance monitoring of the stabilized sites, and to expand the economic evaluation of the RPP technique. The following major tasks were undertaken to address these goals:

- Task 1 – Site Selection: Identified and located potential sites, collected relevant information on the sites, and evaluated the suitability of the sites for demonstration/evaluation of the RPP stabilization technique.
- Task 2 – Design: Analyzed alternative stabilization schemes for the selected sites and selected schemes to be used with due consideration given to both demonstrating the potential effectiveness of the technique while at the same time attempting to optimize the technique using more economical reinforcement layouts.
- Task 3 – Field Installation: Installed reinforcing members according to the selected schemes at each test site.
- Task 4 – Performance Monitoring and Assessment: Developed and installed field instrumentation, and monitored performance of the stabilized slopes at the respective test sites.
- Task 5 – Economic Evaluation: Collected economic data for the stabilized sites to expand the database of costs for applying the technique.
- Task 6 – Continuing Efforts: Continued ongoing efforts initiated in Phase I including development and enhancement of the design methodology and continued performance monitoring of the Emma demonstration site.

This report provides detailed documentation of the activities performed during Phase II to establish the additional test sites, the performance of the sites since installation, as well as an overall evaluation of the technique based on the performance of the sites to date.

1.4. Structure of Report

In general, the report is organized with respect to the different test sites with several additional chapters to describe other pertinent activities and information. The process utilized to select the field test sites is described in Chapter 2. The general design methodology used to date is described in Chapter 3 and a summary of the engineering properties for the recycled plastic members is provided in Chapter 4.

Activities undertaken to establish each of the respective test sites are described in Chapters 5 through 8. Each of these chapters contain general descriptions of the site, a summary of soil properties determined for the sites, a summary of the stability analyses performed, a description of the selected stabilization scheme(s), descriptions of the field installation of reinforcing members and associated instrumentation, and finally a summary of results obtained from the instrumentation since installation was completed.

Chapter 9 contains a summary of the implications drawn from the project to date including discussion of the overall effectiveness of the technique, the suitability of the design method, the construction techniques utilized, and the costs involved with installation of the reinforcing members. Chapter 9 also includes discussion of future widespread implementation of the technique across MoDOT. Finally, Chapter 10 contains a summary of the report and conclusions and recommendations that can be drawn from the project to date.

Chapter 2. Site Selection

A critical task in Phase II of the project was selection of the particular sites to be included in the field testing program. Well over fifty different sites where surficial slope failures had recently occurred were considered for stabilization as a part of Phase II. Candidate sites were identified from a variety of sources including project investigators, MoDOT geotechnical personnel in Jefferson City, and other MoDOT personnel located in districts throughout the state. While the overall objective of site selection was to select sites that would establish the range of site conditions in which recycled plastic reinforcement can be utilized, other criteria were also considered to ensure the success of the project. In this chapter, the criteria and process utilized for selecting the demonstration sites are described along with a brief summary of the characteristics of the sites selected for stabilization during Phase II of the project. Detailed characteristics for each selected site are provided in subsequent chapters.

2.1. Criteria for Site Selection

Selection of the sites to be stabilized during Phase II was a complicated issue. The constructability and performance of the technique is likely to be affected by soil type, slope geometry, stabilization scheme, construction method, and climatic conditions, among many others. Addressing all of these issues with a limited number of additional test sites during Phase II was never possible. Furthermore, the "available" sites at any given time may have characteristics that are better suited to evaluating some of these issues, but not others. Site selection activities therefore focused on selecting sites that would maximize the number of different issues that could be addressed while also addressing the specific issues that were considered to be most important. Specific project constraints such as schedule, budget, and convenience for long-term monitoring also had to be considered.

The criteria considered for selecting the test sites are listed in Table 2.1 along with the issues to be evaluated for each criterion. In general, preference was given to selection of slopes with a range of different geometries and soil types, having both excavated and embankment slopes, and to slides of reasonable size so that different stabilization schemes could be evaluated at a single site while still remaining within the project budget. Other criteria were then considered in a secondary manner.

Table 2.1 Criteria used for evaluation of sites considered for Phase II.

Criteria (Variable)	Issue
Embankment or Cut Slope	Performance
Angle of Slope	Constructability, performance
Soil Type	Constructability, performance
Stratigraphy	Constructability, performance
Depth of Slide	Applicability
Size of Slide Area	Stabilization scheme, Budget, Economics
Presence of Debris in Slope	Constructability
Location of Slope Relative to Pavement	Constructability, Safety
Geographic Location of the Slope	Climate, convenience for monitoring

2.2. Site Selection

Based on these criteria, candidate sites were identified by project and MoDOT personnel. Each of these sites was then screened based on general characteristics such as size, expected soil type, apparent depth of slide, and location. Well over fifty promising sites were then visited by project personnel to photograph the slope, map the surface features, and in some cases collect samples of soil for preliminary classification and testing. More detailed investigations were then performed for the seven sites deemed to be most promising as summarized in Table 2.2.

Table 2.2 Summary of most promising sites considered for stabilization during Phase II.

Site	MoDOT District	Slope Inclination (H:V)	Slope Height (ft)	General Characteristics
I435-Wornall Road	4	2.2:1	32	Embankment with lean clay over clay shale fill
I435-Holmes Road	4	2.2:1	15	Embankment with lean clay over clay shale fill
MO13-Bolivar[1]	8	1.7:1	20	Embankment slope with lean to fat clays
US36-Stewartsville	1	2.2:1	29	Excavated slope with lean clay over fat clay
US54-Fulton	5	3.2:1	46	Excavated slope in "ablation" till
US63-Columbia[1]	5	2.5:1	25	Excavated slope over rock ledge
I44-Sarcoxie[1]	7	2.0:1	24	Excavated slope in gravelly clay
I70-Emma (Phase II)	2	2.5:1	22	Embankment slope with lean to fat clay

[1] site not ultimately selected for stabilization

Five of these sites were ultimately selected for stabilization during Phase II. General locations of the selected sites are shown in Figure 2.1. Two of the selected sites are located on Interstate 435 in Kansas City Missouri. One of the sites is located on U.S. Highway 36 in northwestern Missouri. Another is located on U.S. Highway 54 in the central part of the state. The final test site selected is the I70-Emma site, located on Interstate 70 approximately midway between Columbia and Kansas City in west-central Missouri. This site is the "proof of concept" site that was established in Phase I. As described in more detail in Chapter 7, the original stabilized areas in Phase I have performed well, but two control areas at the site have since failed. This presented a unique opportunity to evaluate several more economical reinforcement schemes at a well characterized site with several years of successful experience using recycled plastic reinforcement.

The selected sites include three embankment slopes and two excavated slopes. Each of the slopes are large enough to represent typical slopes in the State of Missouri. The inclinations of the slopes vary from 2.2H:1V (horizontal:vertical) to 3.2H:1V with heights ranging from 15- to 46-ft (4.5- to 14-m). The soil types generally include lean and fat clays with two of the slopes having layered stratigraphies consisting of a relatively thin surficial layer of lean to fat clay overlying much stiffer fat clay or clay shale. Several of the slopes contain scattered gravel and cobbles and the I70-Emma site contains significant construction debris and rubble fill placed in previous stabilization attempts. More detailed descriptions of each of the selected sites are provided in Chapter 5 through 8.

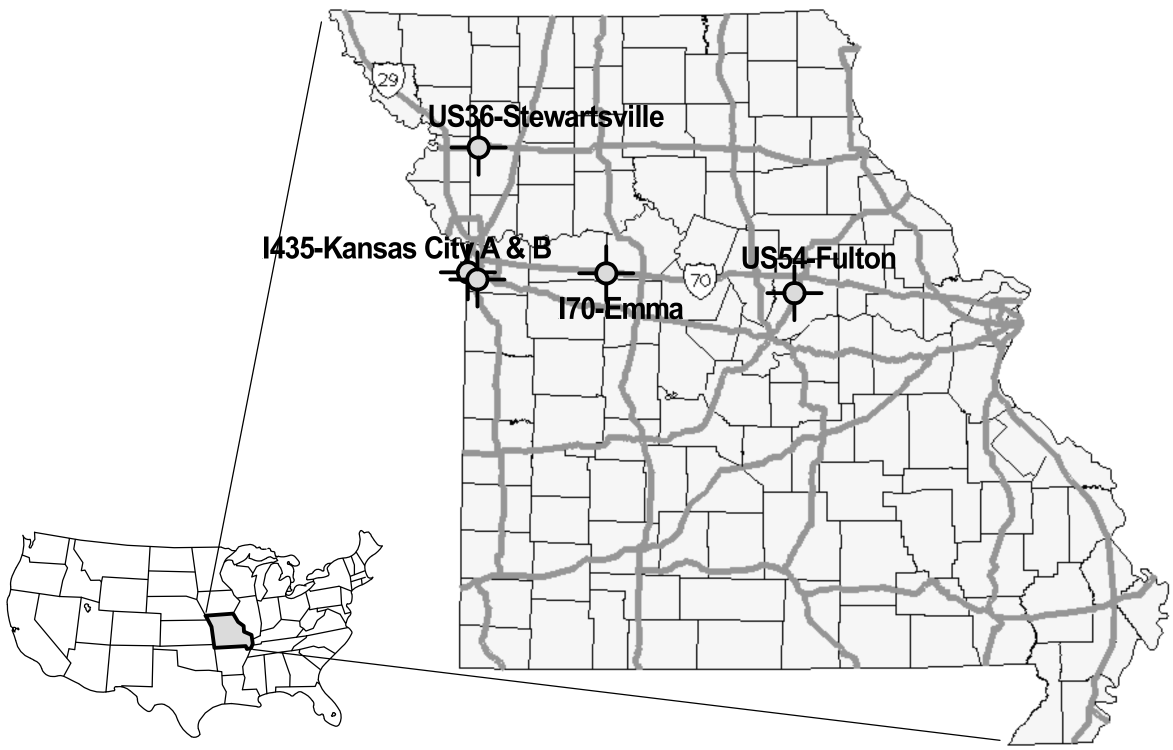


Figure 2.1 Map of the State of Missouri showing locations of the selected stabilization sites.

The reasons for not selecting the remaining sites in Table 2.2 varied. The MO13-Bolivar site was a very attractive site because the embankment contained significant gravel and the slope was very steep, both of which would provide a challenge for stabilization using recycled plastic members. However, the area to be stabilized was larger than could be accommodated within the project budget; additional funding from MoDOT District 8 was therefore needed to stabilize the entire slide area. The roadway was also scheduled for widening in the near future, which meant that the length of time available for monitoring would be limited. Because of these issues, MoDOT District 8 personnel decided to accelerate the construction schedule and simply flatten the slope during the roadway widening project.

The I44-Sarcoxie site was initially also very attractive because of the presence of significant gravel in the surficial soils sampled during the site visit. However, boring and sampling activities revealed that the slope was actually composed predominantly of high plasticity clay soils and, as such, was similar to other sites being considered. The failure also appeared to be relatively deep and the site is located a significant distance from the University of Missouri campus, which meant that field performance monitoring would be difficult. The site was therefore eliminated in favor of other sites. The US63-Columbia site was attractive because of its close proximity to the University, which would permit the site to be frequently monitored. However, other sites seemed to have more significant advantages so the US63-Columbia site was not selected in the final evaluation.

Perhaps the two most significant limitations of the five selected sites is that none of the selected slopes contain significant gravel and none have slope angles greater than 2H:1V. Both of these characteristics would serve as a significant test of the installation method. However, very few slopes with these characteristics were identified during the site selection process, and none of the identified sites could be utilized for the project for varied reasons. However, it is important to note that the lack of a significant number of potential sites with such characteristics is an indication that (1) there are few slopes in Missouri with these characteristics or (2) that few surficial slides occur in slopes with these characteristics. This is not altogether unexpected due to the inherent strength of most gravelly soils and because MoDOT constructs very few permanent slopes steeper than 2:1.

2.3. Summary

In this chapter, the process used to select the five sites for stabilization as a part of Phase II was described. The selected slopes share characteristics with the majority of slopes that experience surficial slides in the State. As such, the effectiveness of the stabilization measures installed at the selected sites is expected to be representative of the effectiveness that can be achieved for most surficial slides within the State. The slopes share some similarities but also have distinct differences, which will allow direct evaluation of the effectiveness of alternative stabilization schemes, e.g., different reinforcement placements, while still evaluating the range of applicability.

Chapter 3. Design Methodology

The general approach taken for analysis of reinforced slopes is to first establish the resistance provided by individual reinforcing members and then to incorporate that resistance into classic slope stability analysis procedures to determine the factor of safety for the reinforced slope. Given the resistance provided by individual reinforcing members, the mechanics of stability analyses incorporating these forces is relatively well established. Development of the distribution of resistance provided by a single member is less well established. In this chapter, the general approach taken to analyze the stability of slopes reinforced with recycled plastic and "strong" reinforcing members is described with particular focus on development of the distribution of resisting force along reinforcing members. The developed method utilizes a limit state design approach that considers a series of potential failure modes.

3.1. General Approach to Stability Analysis

The general approach adopted for evaluating the stability of reinforced and unreinforced slopes is to first assume a potential sliding surface and then calculate a factor of safety for that sliding surface based on consideration of the equilibrium of the free body formed by the sliding surface and slope surface as shown in Figure 3.1. For most slope stability analyses, the factor of safety, F, is defined as

$$F = \frac{\int s}{\int \tau} \qquad (3.1)$$

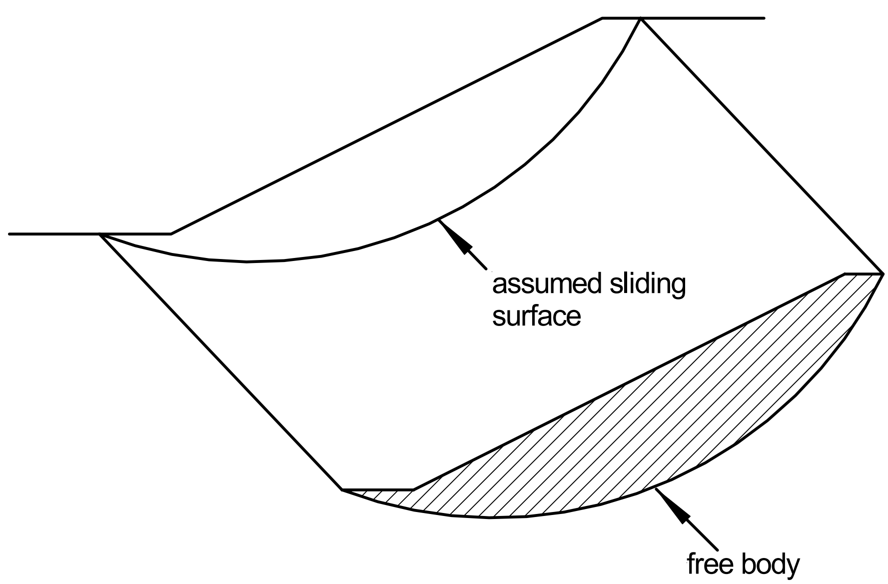


Figure 3.1 Free body diagram considered for equilibrium in slope stability analysis.

where s is the available shear strength and τ is the mobilized shear stress on the assumed sliding surface. In the general case, the available shear strength (s) is a function of the normal stress, σ, on the sliding surface and is often expressed using the Mohr-Coulomb failure criterion

$$s = c + \sigma \tan \phi \tag{3.2}$$

where c is the cohesion intercept and ϕ is the angle of internal friction for the soil on the sliding surface. In terms of effective stresses, the Mohr-Coulomb criterion is expressed as

$$s = \bar{c} + (\sigma - u)\tan \bar{\phi} = \bar{c} + \bar{\sigma} \tan \bar{\phi} \tag{3.3}$$

where u is the pore pressure on the sliding surface, $\bar{\sigma}$ is the effective stress on the sliding surface, and $\bar{c}$ and $\bar{\phi}$ are respectively the cohesion intercept and angle of internal friction expressed in terms of effective stresses. Substituting Equation 3.3 into Equation 3.1 results in the following expression for the factor of safety in terms of effective stresses

$$F = \frac{\int \left(\bar{c} + (\sigma - u)\tan \bar{\phi}\right)}{\int \tau} = \frac{\int \left(\bar{c} + \bar{\sigma} \tan \bar{\phi}\right)}{\int \tau} \tag{3.4}$$

Equation 3.4 indicates that the factor of safety along an assumed sliding surface is dependent on (1) the Mohr-Coulomb strength parameters ($\bar{c}$ and $\bar{\phi}$) for the soil on the sliding surface, (2) the normal stress (σ) on the sliding surface, (3) the pore pressure (u) on the sliding surface and (4) the mobilized shear stress (τ) on the sliding surface. The Mohr-Coulomb strength parameters ($\bar{c}$ and $\bar{\phi}$) and the pore pressure (u) are assumed to be known. The distribution of normal stress (σ) and shear stress (τ) along the potential sliding surface are unknown and must be determined from equilibrium of the sliding body.

The most common approach to determine the distribution of normal and shear stress is to use a method of slices as depicted in Figure 3.2. In this approach, the sliding body is divided into a number of vertical slices and equilibrium of the individual slices is considered to determine the normal and shear forces (or stresses) on the sliding surface and the factor of safety for an assumed sliding surface. The process is then repeated for other potential sliding surfaces until the most critical sliding surface – the surface giving the lowest value of the factor of safety – is found. The factor of safety associated with the most critical sliding surface is taken to represent the stability of the slope.

A similar approach is adopted for reinforced slopes except that a force due to a reinforcing member, F_R, is added to the other forces on the slices that are intersected by reinforcing members as shown in Figure 3.3. This force is included in development of equilibrium equations that are used to solve for the overall factor of safety for the slope. It is important to point out that the reinforcement force (F_R) may have components both perpendicular and parallel to the reinforcing member and that F_R is considered a known quantity and must be provided for the stability analysis.

The reinforcement force modifies the factor of safety in several ways. First, the reinforcement force provides a direct resistance to sliding. This direct resistance will always tend to increase the factor of safety over that for the unreinforced slope. In addition, the reinforcement force can modify the computed normal and shear forces on the sliding surface and thereby change the factor of safety as compared to an unreinforced slope. These forces can either increase or decrease the factor of safety depending on the inclination of the reinforcement force (F_R) with respect to the sliding surface and the respective magnitudes of the axial and lateral components of the reinforcement force. Note that for limit equilibrium analyses, forces due to reinforcement are generally taken as the maximum resisting force that can be developed for the reinforcing element. The forces are therefore referred to as "limit resistances" in this report.

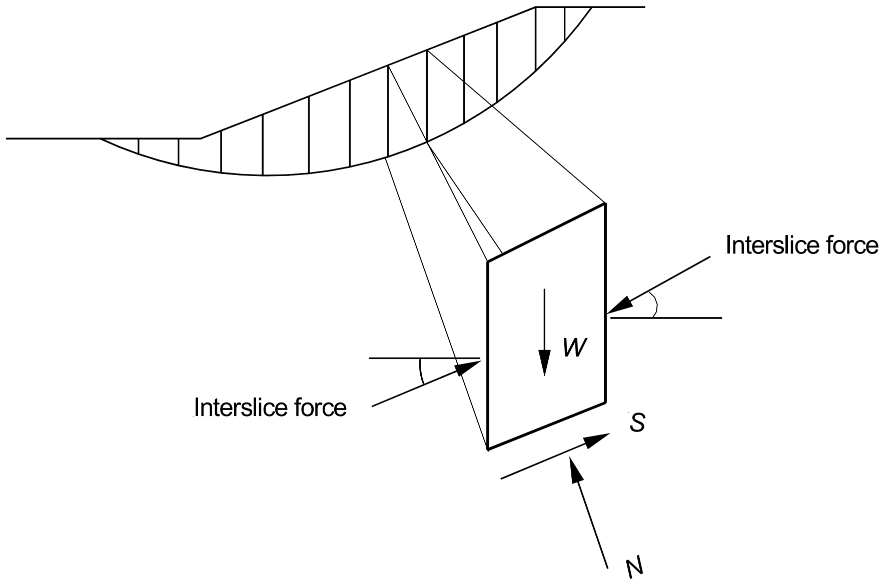


Figure 3.2 Static equilibrium of individual slice in the Method of Slices.

In general, the magnitude of the resisting force that is included in the stability analysis varies with position along the reinforcing member. The distribution of the reinforcement force is described by a "limit resistance curve" as shown conceptually in Figure 3.4. The limit resistance curve defines the magnitude of the resisting force provided by the reinforcing member as a function of the location where a potential sliding surface crosses the member. As illustrated in Figure 3.5, each reinforcing member on a slope will provide a resisting force based on the location of the intersection of the sliding surface and the reinforcing member. The method adopted for computing the limit resistance distribution for reinforcing members is described in the following section.

3.2. Development of Limit Lateral Resistance Curves

A method for predicting the limit lateral resistance of individual reinforcing members has been developed. The method uses a limit state design approach wherein a series of potential failure mechanisms are considered in developing the overall distribution of lateral resistance along a reinforcing member. The procedure is based on consideration of the following limit states:

- failure of soil around or between reinforcing members – referred to as the "limit soil resistance",
- structural failure of reinforcing members in shear or bending due to loads applied from the soil mass – referred to as the "limit member resistance", and

- failure of soil due to insufficient anchorage length – referred to as the "limit anchorage resistance".

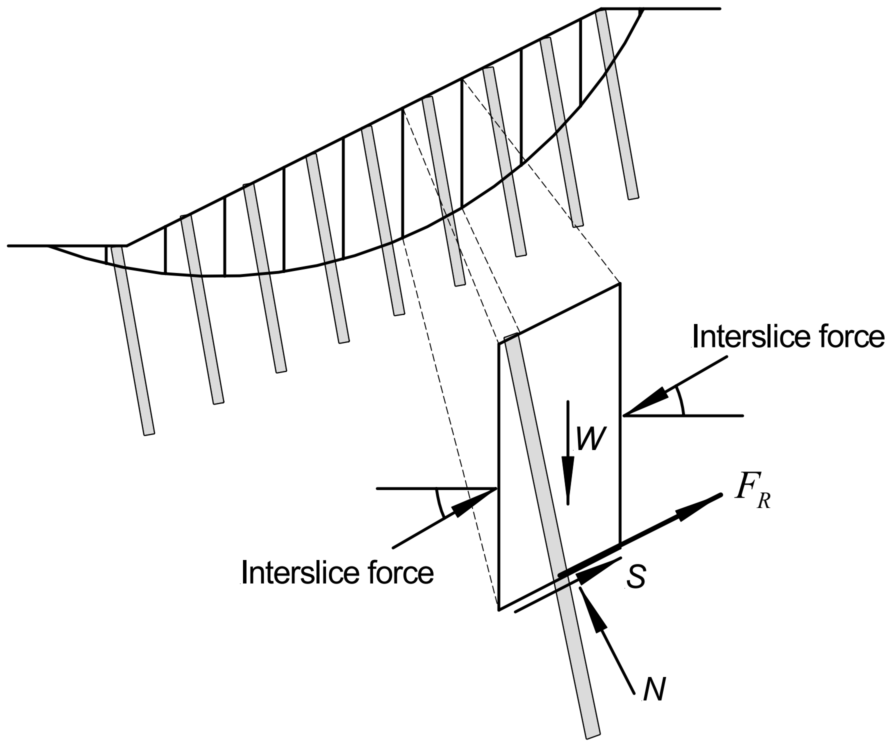


Figure 3.3 Reinforcement force (F_R) on an individual slice in the Method of Slices.

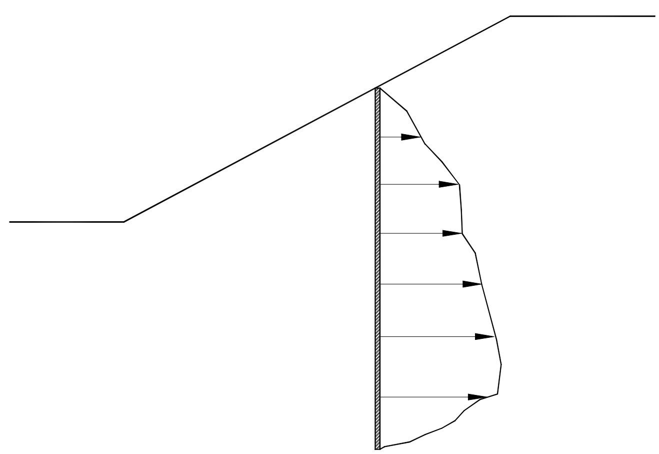


Figure 3.4 Conceptual distribution of limit resistance along a reinforcing member in a slope.

In the method, separate limit resistance curves are developed for each limit state as illustrated in Figure 3.6. From these individual curves, a "composite" limit resistance curve that corresponds to the most critical component of resistance at each sliding depth is established by taking the component with the least resistance at each sliding depth. The resulting composite limit resistance curve obtained in this manner is shown in Figure 3.7. Each of the limit states considered in the design are discussed in the following sections.

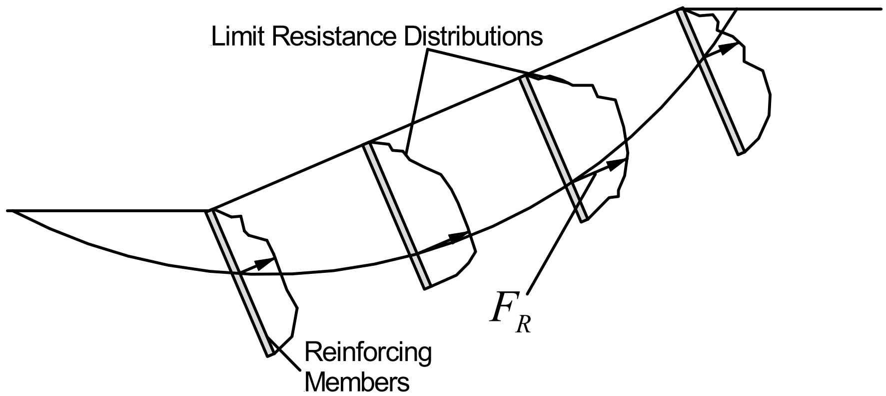


Figure 3.5 Example of distributions of limiting resistance for multiple members in a reinforced slope.

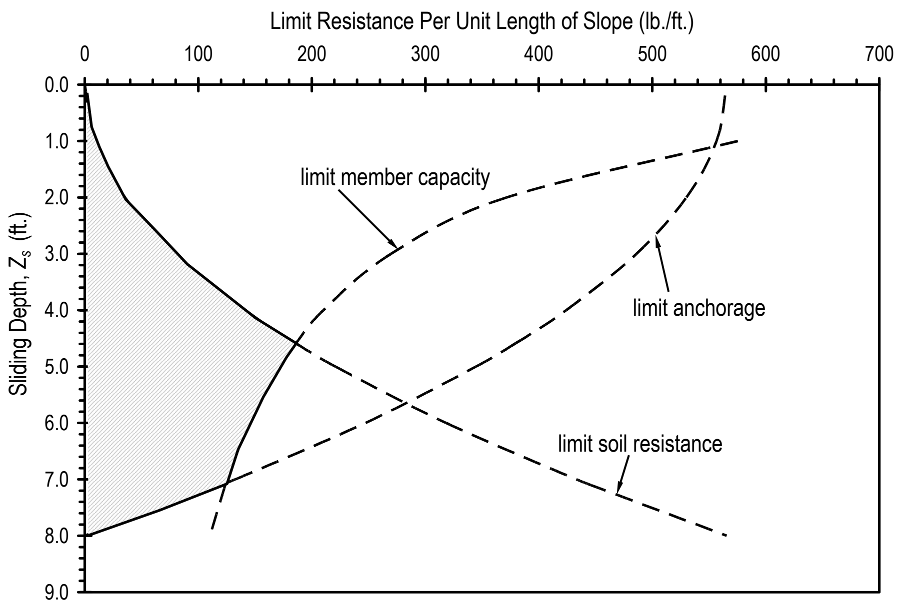


Figure 3.6 Typical distributions of limit resistance developed for the three limit states considered.

3.2.1. Limit Soil Resistance

The first limit state considered is the one for failure of soil around or between reinforcing members, which produces the “limit soil resistance”. Calculation of the limit soil resistance requires that the lateral pressure at which failure of the soil will occur be known. This pressure is referred to as the “limit soil pressure” and is denoted p_u. Several alternative methods have been proposed for predicting the limit soil pressure for stabilizing piles (e.g. Broms, 1964; Reese, 1974). For the current work, the method proposed by Ito and Matsui (1975) was selected over other methods because it is flexible enough to be extended to members composed of non-conventional materials and because it is considered one of the more conservative of the available methods for typical member spacings. Other methods available for predicting the limit soil pressure are generally based on load tests for full-scale conventional steel and concrete piles, which are considerably different in size and stiffness than the recycled plastic members of primary concern in this report.

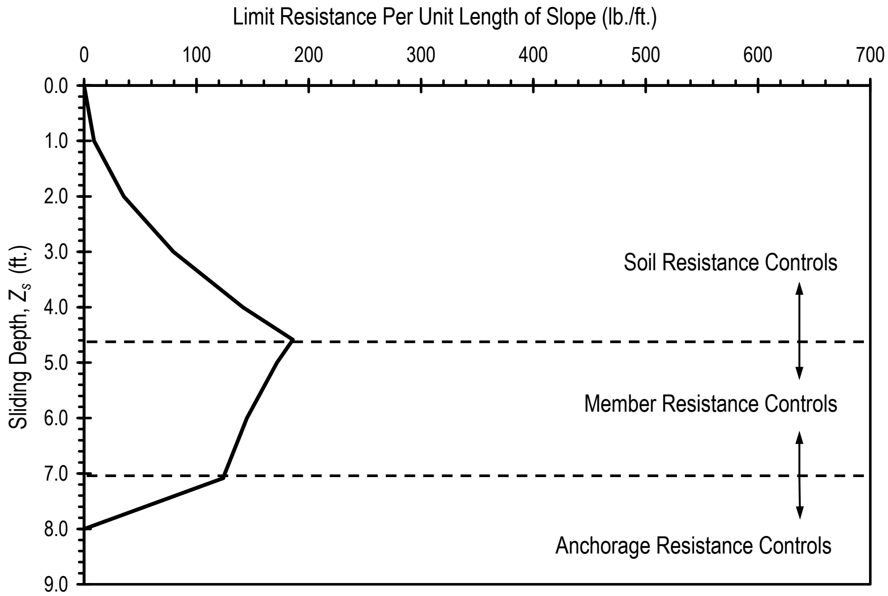


Figure 3.7 Typical "composite" limit lateral resistance curve obtained by considering: failure of the soil surrounding the reinforcing member, failure of the reinforcing member, and failure due to insufficient anchorage length.

The Ito and Matsui theory is based on calculation of the net lateral force acting on a row of stabilizing piles due to the surrounding ground undergoing plastic deformation as illustrated in Figure 3.8. The lateral forces are derived from the theory of plastic deformation where the soil between adjacent piles in a row is assumed to be in a state of failure for the Mohr-Coulomb yield criterion as shown by the shaded area (AEBB'E'A') in Figure 3.9. It is assumed that plain strain conditions exist along the length of the pile, the pile is infinitely long, and the pile is rigid with respect to the surrounding soil. Based on this theory, the force per unit length acting on the pile at depth *z* below the ground surface is given by (Matsui et al., 1982):

$$P_u(z) = cD_1\left(\frac{D_1}{D_2}\right)^{G_1(\phi)}\left[\frac{2N_\phi^{1/2}\tan\phi + 1}{N_\phi \tan\phi}\left\{e^{\left(\frac{D_1 - D_2}{D_2}G_3(\phi)\right)} - 1\right\} + \frac{G_2(\phi)}{G_1(\phi)}\right] - cD_1\frac{G_2(\phi)}{G_1(\phi)} + \sigma_h\left\{D_1\left(\frac{D_1}{D_2}\right)^{G_1(\phi)} e^{\left(\frac{D_1 - D_2}{D_2}G_3(\phi)\right)} - D_2\right\} \tag{3.5}$$

where $N_\phi = \tan^2\left(\pi/4 + \phi/2\right)$, $G_1(\phi) = N_\phi^{1/2}\tan\phi + N_\phi - 1$, $G_2(\phi) = 2\tan\phi + 2N_\phi^{1/2} + N_\phi^{-1/2}$, $G_3(\phi) = N_\phi \tan\phi \tan\left(\pi/8 + \phi/4\right)$, D_1 is the center-to-center spacing between reinforcing members, D_2 is the edge-to-edge distance between reinforcing members, c and ϕ are the cohesion intercept and angle of internal friction for the soil, and σ_h is the lateral earth pressure acting on A-A' in Figure 3.9 (which is generally taken to be the active earth pressure). In the case of saturated soil under undrained loading conditions, the strength of the soil will be purely cohesive ($\phi = 0$) and Equation 3.5 reduces to the form

$$P_u(z) = cD_1\left(3\ln\frac{D_1}{D_2} + \frac{D_1 - D_2}{D_2}\tan\frac{\pi}{8}\right) + \sigma_h\left(D_1 - D_2\right) \tag{3.6}$$

For cohesionless soil ($c = 0$), Equation 3.5 reduces to the form

$$P_u(z) = \sigma_h\left\{D_1\left(\frac{D_1}{D_2}\right)^{G_1(\phi)} e^{\left(\frac{D_1 - D_2}{D_2}G_3(\phi)\right)} - D_2\right\} \tag{3.7}$$

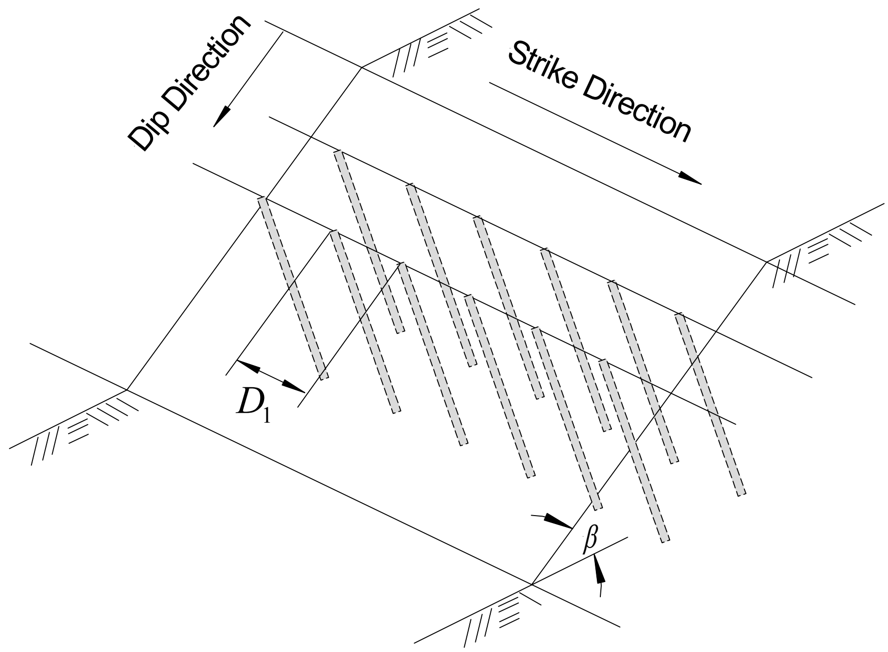


Figure 3.8 Slope reinforced with a single row of reinforcing members spaced equally in the longitudinal (strike) direction.

A typical plot of limit soil pressure (P_u) obtained from the Ito and Matsui method is shown in Figure 3.10. As shown, the limit soil pressure increases linearly with depth. It is important to note that the limit lateral soil pressure at the ground surface is zero, i.e. $P_u(0)=0$, if $c=0$ and is not zero, i.e. $P_u(0)\neq0$, if $c\neq0$.

The limit soil pressure is the pressure that will cause the soil to fail laterally at a particular depth. If it is assumed that this load can be simultaneously mobilized along the length of the reinforcing member above the sliding surface, the total limit resistance force based on failure of soil above the sliding surface is obtained by integrating the computed limit soil pressure over the length of reinforcement above the sliding surface as shown in

Figure 3.11. For stability analysis, this total limit resistance force is assumed to act at the sliding surface as shown in Figure 3.11b.

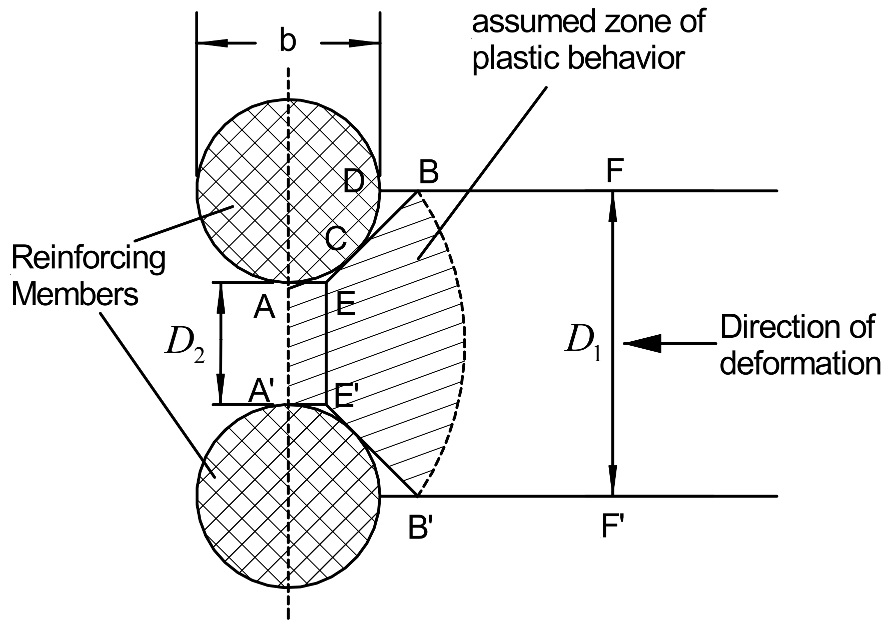


Figure 3.9 Assumed zone of plastic behavior between adjacent piles in a row (after Ito and Matsui, 1975).

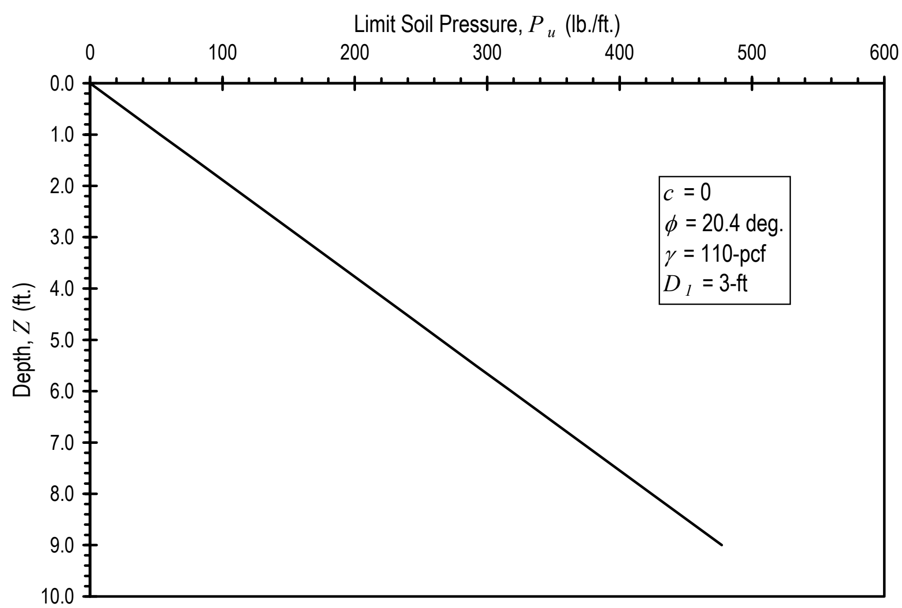


Figure 3.10 Typical plot of limit soil pressure (P_u) from Ito and Matsui plastic deformation theory for cohesionless soil.

Since the sliding surface may in general pass through any point on the reinforcing member, additional points on the limit resistance curve are computed by repeating the integration for different sliding depths to establish a complete limit resistance curve describing the total resistance as a function of sliding depth as shown in Figure 3.12. The total resistance increases from a minimum value at the ground surface to a maximum value at the end of the reinforcing member. Since stability analyses are generally performed for a cross-section of unit width, the total resisting forces computed by integrating the limit soil pressure are divided by the longitudinal spacing (D_l) to produce values of the limit force per unit width suitable for stability analyses.

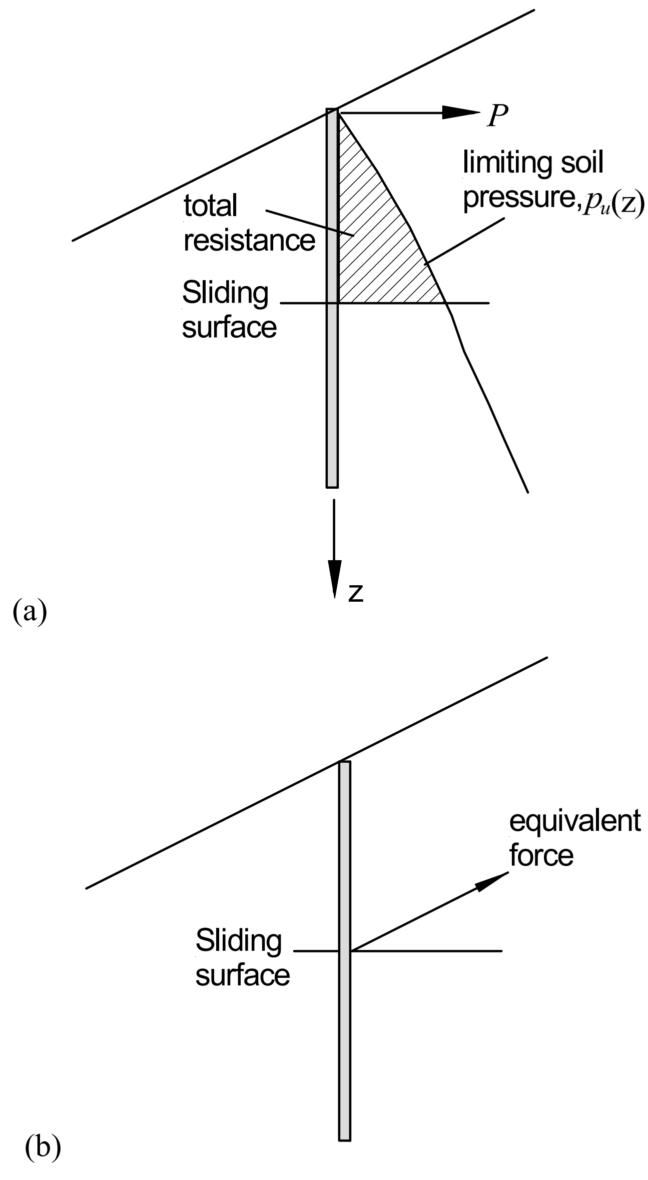


Figure 3.11 Graphical illustration of method for computing limit soil resistance: (a) integral of limiting soil pressure, and (b) equivalent total resisting force.

3.2.2. Limit Anchorage Resistance

The second limit state considered is the one in which reinforcing members have insufficient anchorage length beyond the sliding surface to prevent passive failure of the soil adjacent to the members below the sliding surface. If it is assumed that the limiting soil pressure in the passive mode below the sliding surface can also be predicted by Equation 3.5, a similar procedure can be used to calculate the limiting anchorage resistance. The resisting force provided by the length of the reinforcing element extending below the sliding surface can be obtained by integrating the limiting soil pressure (p_u) over the length of the reinforcing element extending from the sliding surface to the end of the member as shown by the shaded zone in Figure 3.13a. It is again assumed that the full limiting soil pressure can be mobilized over the entire length of reinforcing member below the sliding surface. The total resisting force for a particular sliding depth is again replaced with an equivalent force for stability analysis (Figure 3.13b). The complete limiting resistance distribution for the anchorage limit state is calculated by computing the total resisting force for different sliding depths. An example of limiting resistance due to the anchorage length is shown in Figure 3.14. As shown in Figure 3.14, the limiting resistance for the anchorage length increases from zero for a sliding surface passing through the lower end of the reinforcement to a maximum for very shallow sliding surfaces.

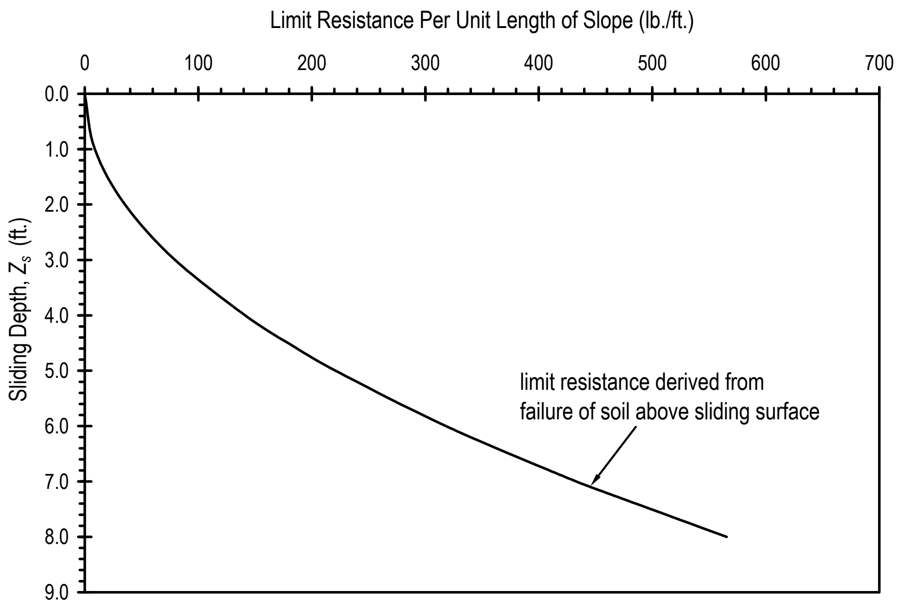


Figure 3.12 Typical limit soil resistance curve.

A "composite" limit resistance curve can be developed that considers limit states for both failure of the soil above the sliding surface and failure due to inadequate anchorage length by simply superimposing the limit resistance curves for the two limit states (Figures 3.12 and 3.14) and taking the least of the two limit resistances at each sliding depth as shown in Figure 3.15. The resulting composite limit resistance curve is shown in Figure 3.16. As shown in the figure, the composite limit resistance considering both failure of the soil above the sliding surface and anchorage failure below the sliding surface increases from zero at the ground surface to a maximum value at an intermediate depth and then decreases back to zero at the end of the reinforcing member. As discussed in Section 3.2.1, the total resisting force is divided by the member spacing (D_1) since stability analyses are generally performed for a cross-section of unit width. The limit resistance plotted in Figure 3.16 does not consider the potential for structural failure of the reinforcing member and is therefore only suitable for "strong" reinforcing members that have sufficient capacity to take the loads that may be imposed on the member if the lateral soil pressure used in developing the limit resistance are applied to the member. The following section describes how this resistance distribution is modified for use with "weak" reinforcing members that have insufficient capacity to resist such loads.

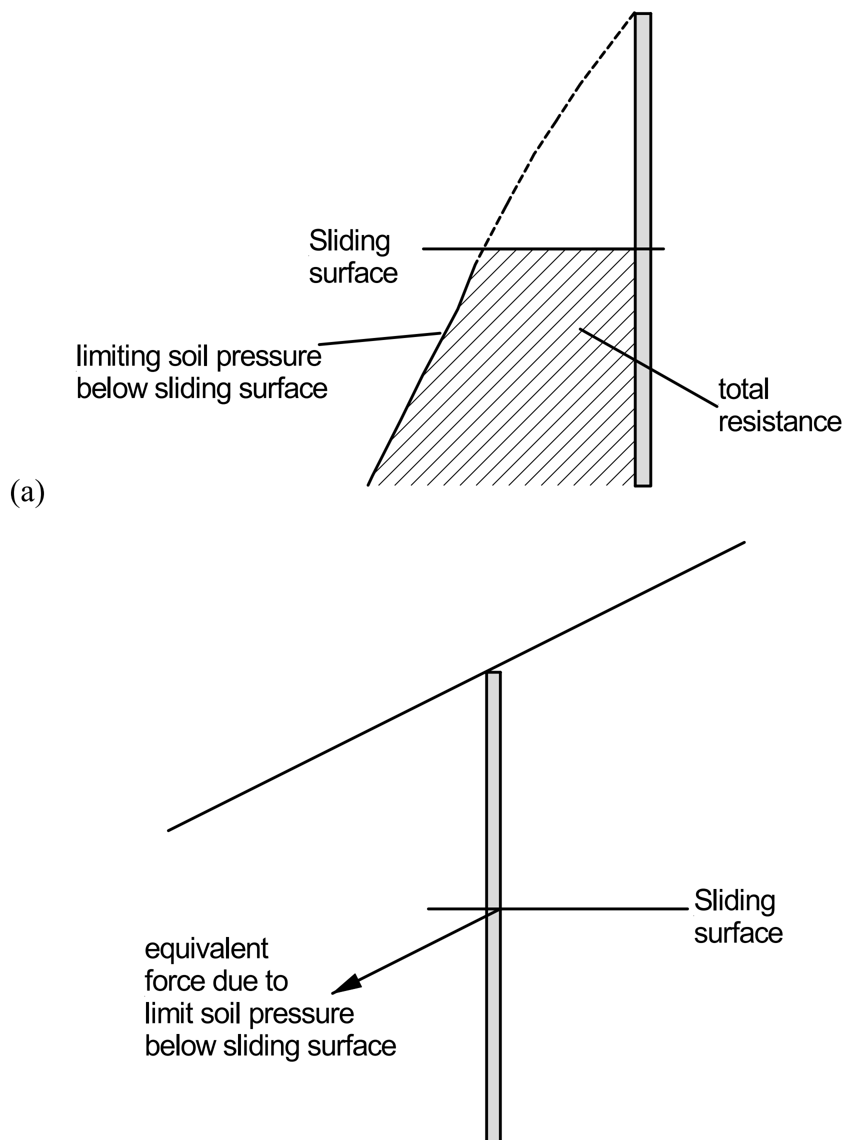


Figure 3.13 Graphical illustration for computing limit anchorage resistance: (a) integral of limiting soil pressure, and (b) equivalent total resisting force.

3.2.3. Limit Member Resistance

The final limit state considered is structural failure of reinforcing members in bending or shear. Application of the predicted limit lateral soil pressures used for development of the previous limit resistance curves may lead to bending moments or shear forces that exceed the capacity of the reinforcing member. In this case, the member is expected to fail prior to the limit soil pressures being fully mobilized and the stabilizing forces predicted by considering failure of the soil alone will be unconservative. It is therefore important to consider the capacity of reinforcing members in both shear and bending in developing the final limit resistance curves.

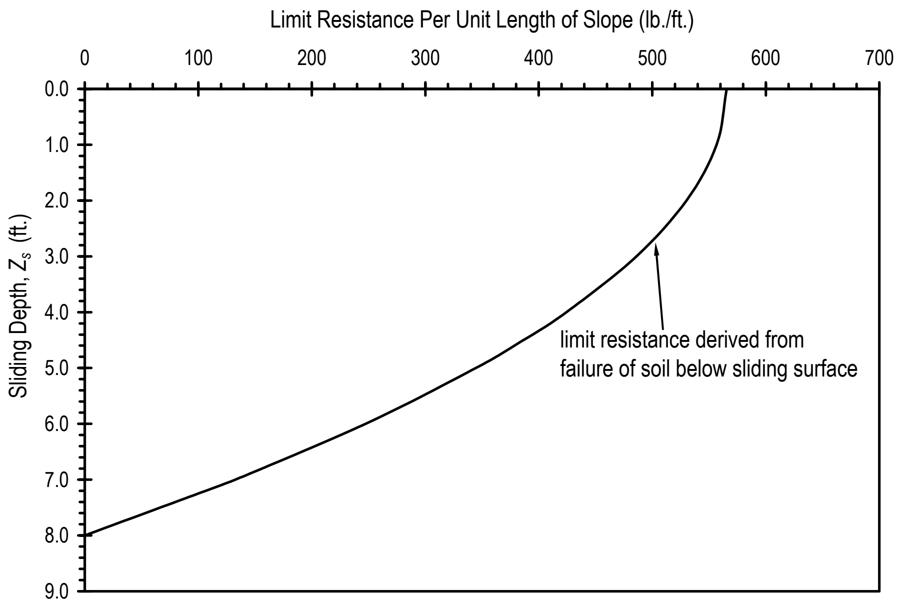


Figure 3.14 Typical limit anchorage resistance curve.

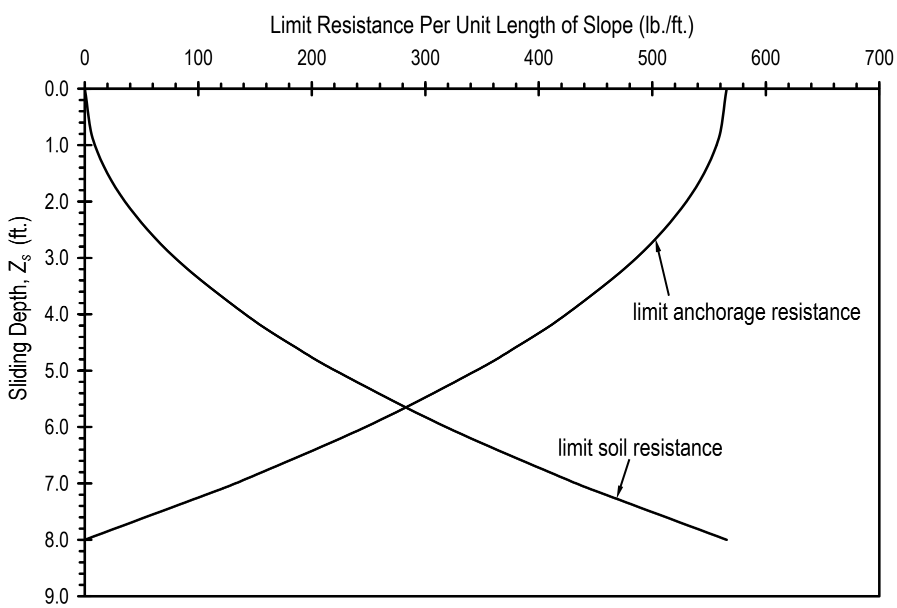


Figure 3.15 Limit resistance curves considering failure of soil around the reinforcing member and anchorage failure.

The approach used to account for the potential of the reinforcing member to fail structurally is to consider a factored pressure distribution of the form

$$p'(z) = \alpha \, p_u(z) \tag{3.8}$$

where $p'(z)$ is a factored pressure distribution and $p_u(z)$ is the limit soil pressure. The unknown factor α is the factor that will produce a distribution of soil pressures ($p'(z)$) such that the mobilized maximum shear or moment just equals the shear or moment capacity of the reinforcing member, respectively. Once the factor α is determined, the resistance force is computed by integrating the factored pressure distribution in a manner similar to that used for the other limit states considered. Since the distribution of shear and moment, and the maximum shear and moment, are functions of the sliding depth, the factor α must also be a function of the sliding depth. Separate factors, α_s and α_m, are computed for shear and moment, respectively, and the lower of the two factored pressure distributions is used to compute the limiting resistance for each sliding depth.

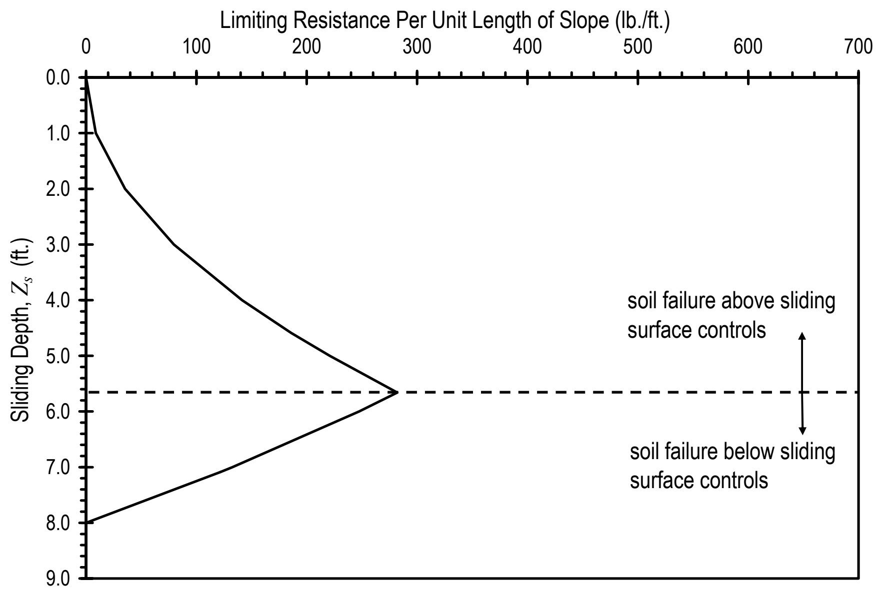


Figure 3.16 Composite limit resistance curve considering failure of soil around the reinforcing members and anchorage failure.

The procedure for determining the limiting resistance distribution for structural failure of the reinforcing member is as follows:

(1) Assume a depth of sliding;

(2) Compute the maximum shear and moment in the reinforcing member when subjected to the limiting soil pressure (p_u) above the depth of sliding;

(3) Compute the factors α_s and α_m that produce a factored pressure distribution $p'(z)$ that will mobilize maximum shear and moment equal to the shear and moment capacity of the reinforcing member;

(4) Compute the limit resistance for the assumed sliding depth by integrating the factored soil pressure $p'(z)$ from the top of the reinforcing member to the sliding depth; and

(5) Assume additional sliding depths and repeat steps (2) through (4) to get the distribution of limit resistance with sliding depth.

The following sections describe the method used to compute the maximum shear and moment along the reinforcing member for different sliding depths and the method for computing the factors α_s and α_m.

Calculation of factored pressure distribution considering shear. The distribution of shear forces along the reinforcing member is calculated from elastic analysis as is commonly done for laterally loaded piles. In this analysis, it is assumed that the lateral load above the sliding surface is equal to the limit soil pressure. The lateral load below the sliding surface is determined from elastic analysis considering the stiffness of the reinforcing member and the soil surrounding the reinforcing member. For the case of a homogenous soil profile with the reinforcing member subjected to the unfactored limiting soil pressures (p_u) above the sliding surface, the distribution of the shear force in the reinforcing member above the sliding surface is (Ito et al., 1981)

$$[S]_{\bar{z}} = E_p I_p \left\{ 6a_3 + \frac{f_1}{E_p I_p}\bar{z} + \frac{f_2}{2E_p I_p}\bar{z}^2 \right\} \tag{3.9}$$

where E_pI_p is the bending stiffness of the member, $\bar{z}$ is the distance from the sliding surface (positive downward), a_3 is given by

$$a_3 = \frac{H'}{12E_p I_p}(2f_1 - H' f_2) \tag{3.10}$$

f_1 and f_2 are polynomial constants describing the distribution of lateral pressure from Equation 3.5, and H' is the length from the pile top to the sliding surface. The distribution of shear below the sliding surface is computed by assuming that the deflection at the lower tip of the reinforcing member is zero. The distribution of shear is computed as

$$[S]_{\bar{z}} = E_p I_p \left\{ 2\beta^3 e^{-\beta\bar{z}} \left\{ (A+B)\cos\beta\bar{z} - (A-B)\sin\beta\bar{z} \right\} \right\} \tag{3.11}$$

where $\beta = \sqrt[4]{E_s/(4E_p I_p)}$, A and B are integral constants determined by the pile-head fixity condition and continuity of pile at the sliding surface, and E_s is the soil modulus. For members installed without restraint at the surface, the free head condition applies and the integral constants are given by

$$A = \frac{H'}{12E_p I_p \beta^3}\left\{ 3(2+\beta H')f_1 - H'(3+2\beta H')f_2 \right\} \tag{3.12}$$

$$B = \frac{-(H')^2}{12E_p I_p \beta^2}\left(3f_1 - 2H' f_2\right) \tag{3.13}$$

where all terms are defined above. A sample distribution of shear along a reinforcing member computed using Equations 3.9 and 3.11 is shown in Figure 3.17 for the following conditions: c=0, ϕ=31 degrees, γ=110-pcf (17.3-kN/m^3), longitudinal spacing D_l=3-ft (0.9-m), sliding depth Z_s=5-ft (1.5-m), and E_pI_p=19.2-kip-ft^2 (7.9-kN-m^2). For non-homogenous profiles, a similar approach is adopted utilizing numerical methods to determine the distribution of shear in the member.

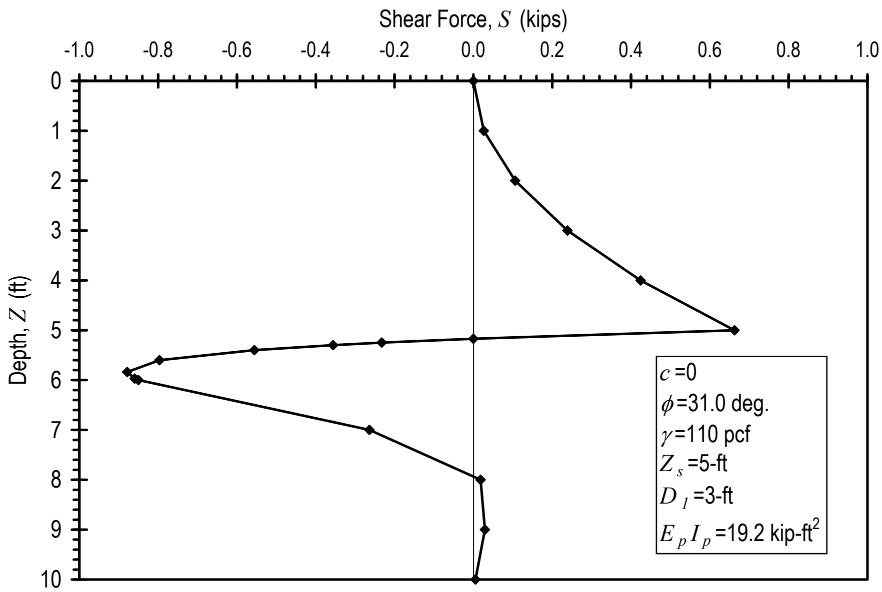


Figure 3.17 Sample distribution of shear along a reinforcing member.

The location of maximum shear force varies with the depth of the sliding surface. It may be located at the depth of sliding or below the sliding depth. Thus, a trial and error approach is used varying the depth below the sliding depth. The maximum shear force (S_{max}) is obtained from shear distribution diagrams (Figure 3.17) as the absolute value of the maximum force along the reinforcing member.

Once the maximum shear mobilized due to the limit soil pressures is known, the factor α_s is approximated as

$$\alpha_s = \frac{S_{ult}}{S_{\max}} \tag{3.14}$$

where S_{ult} is the shear capacity of the reinforcing member. While Equation 3.14 is generally only approximate, results of comparative analyses to date indicate the approximation produces α-factors very close to values determined more rigorously.

The factor α_s is then applied to the limiting lateral pressure distribution to determine the factored pressure distribution to ensure the reinforcing member will not fail in shear. For instance, if the maximum shear obtained from Figure 3.17 is 0.88-kips (3.9-kN), the factor α_s is calculated to be 10.2 for a 4-inch (100-mm) square reinforcing member with a shear capacity of 9-kips (40-kN). The factored pressure distribution for this case is shown in Figure 3.18. Hence, the factored shear diagram (Figure 3.19) corresponding to the required factor α_s is recalculated from Figure 3.18. The factored pressure distribution for shear is computed for each sliding depth and the limiting resistance curve for shear (Figure 3.20) is obtained by integrating the factored pressure distribution for each sliding depth.

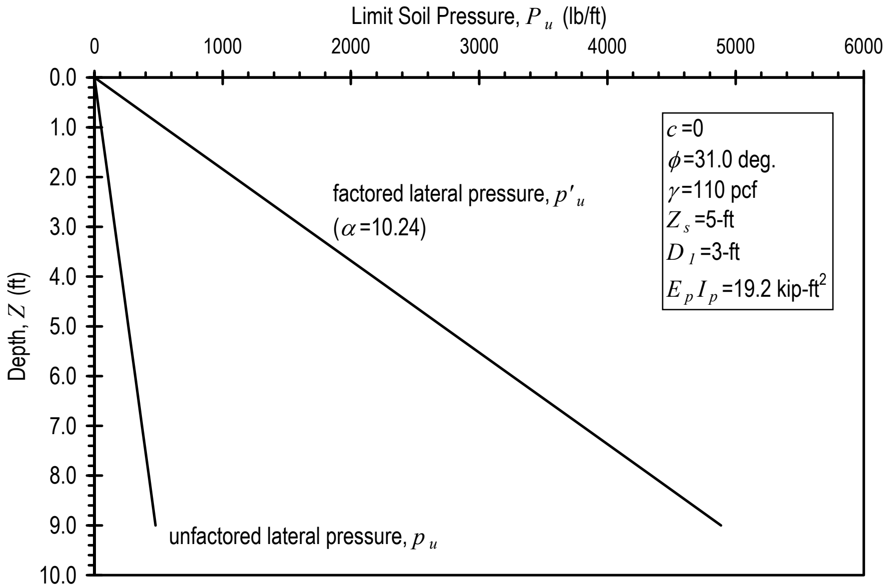


Figure 3.18 Factored pressure distribution considering shear capacity of reinforcing member.

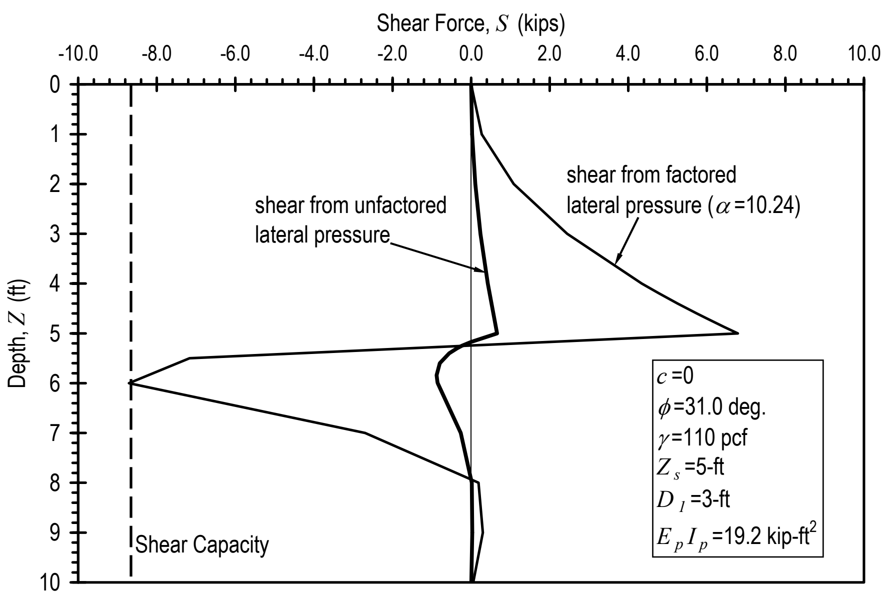


Figure 3.19 Factored and unfactored shear distribution diagrams.

As shown in Figure 3.20, the limiting resistance for shear is constant at shallow sliding depths as the maximum shear occurs at the depth of sliding. The limiting resistance for shear then decreases with sliding depth for deeper sliding depths where the maximum shear occurs at some depth below the sliding depth. As described in Section 3.2.1 and 3.2.2, the integrated total resistance is divided by the member spacing since the stability analyses are generally performed for a cross-section of unit width.

Calculation of factored pressure distribution considering moments. The distribution of moment along reinforcing members is calculated based on similar assumptions. Following the work of Ito and Matsui (1981), the distribution of moment above the sliding surface is given by

$$[M]_{\bar{z}} = E_p I_p \left\{ 2a_2 + 6a_3\bar{z} + \frac{f_1}{2E_p I_p}\bar{z}^2 + \frac{f_2}{6E_p I_p}\bar{z}^3 \right\} \tag{3.15}$$

where a_2, which depends on the continuity of pile at the sliding surface, is taken to be

$$a_2 = \frac{(H')^2}{12E_p I_p}\left(3f_1 - 2H' f_2\right) \tag{3.16}$$

and all other terms are defined previously. The distribution of moment below the sliding surface is given by

$$[M]_{\bar{z}} = E_p I_p \left\{ 2\beta^2 e^{-\beta\bar{z}} (A \sin\beta\bar{z} - B\cos\beta\bar{z}) \right\} \tag{3.17}$$

where A and B are again integral constants for members installed without restraint at the surface presented in Equations 3.12 and 3.13, respectively. A typical moment distribution diagram shown in Figure 3.21 depicts the location of maximum moment for a reinforcing member with a sliding depth Z_s=5-ft (1.5-m), c=0, ϕ=31 degrees, γ=110-pcf (17.3-kN/m^3), D_1=3-ft (0.9-m), and E_pI_p=19.2-kip-ft^2 (7.9-kN-m^2).

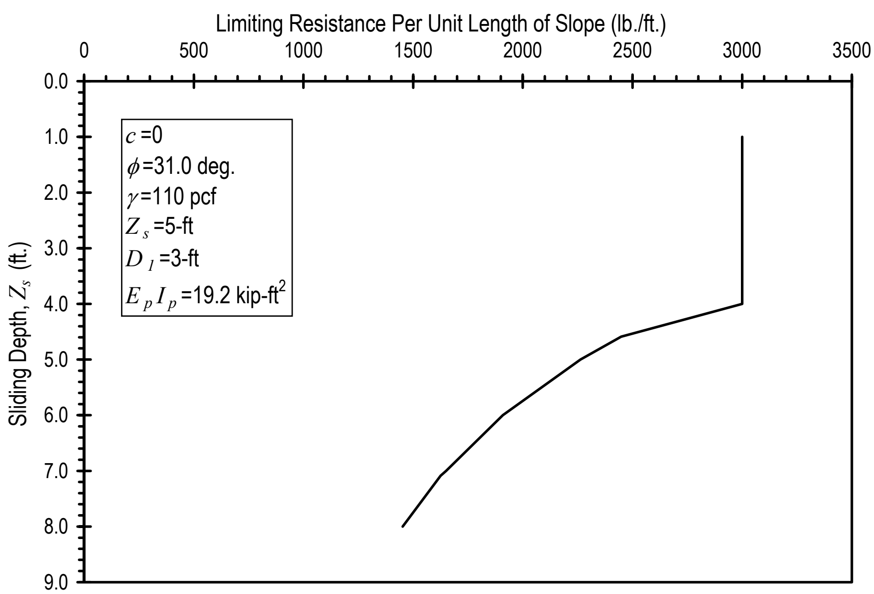


Figure 3.20 Limiting resistance curve considering shear capacity of reinforcing member.

The location of maximum moment also varies with the depth of the sliding surface. The maximum moment (M_{max}) is located just below the sliding surface at the depth where the shearing force becomes zero and is calculated by substituting $\bar{z}_2$ for $\bar{z}$ in Equation 3.17 as shown below

$$[M]_{\bar{z}_2} = E_p I_p \left\{ 2\beta^2 e^{-\beta \bar{z}_2} (A \sin \beta \overline{z_2} - B \cos \beta \overline{z_2}) \right\} \tag{3.18}$$

where $\overline{z_2}$ is given by

$$\overline{z_2} = \frac{1}{\beta} \tan^{-1} \frac{A+B}{A-B} \tag{3.19}$$

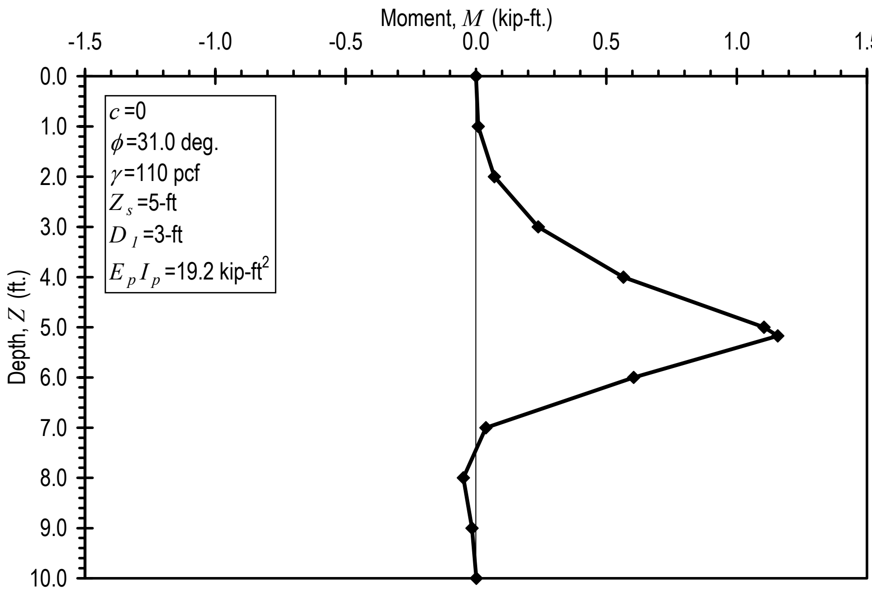


Figure 3.21 Sample distribution of moment along a reinforcing member.

Similar to the method used to calculate α_s, the factor α_m is determined as

$$\alpha_m = \frac{M_{ult}}{M_{\max}} \tag{3.20}$$

where M_{max} is the maximum moment determined from Equation 3.18. The factor α_m is then applied to the limiting soil pressure distribution to determine the factored pressure distribution to avoid structural failure of the reinforcing member in bending. For the conditions described for Figure 3.21, the maximum moment computed from Equation 3.18 is approximately 1.16-kip-ft (1.6-kN-m). Thus, the factor α_m required to ensure the stability of a 4-inch (100-mm) square reinforcing member with a moment capacity of 0.9-kip-ft (1.2-kN-m) is calculated to be 0.78. The factored pressure distribution curve for this case is shown in Figure 3.22. The factor α_m is applied to Figure 3.21 and the factored moment distribution diagram is shown in Figure 3.23. The factored pressure distribution for moment is computed for each sliding depth and the limiting resistance curve for moment (Figure 3.24) is obtained by integrating the factored pressure distribution curve for each sliding depth. As described in Sections 3.2.1 and 3.2.2, the integrated total resistance is divided by the member spacing since stability analyses are generally performed for a cross-section of unit width.

As described above, the method used to calculate the factor α_m is strictly limited to c=0 conditions and is not generally applicable for ϕ=0 and c-ϕ soil conditions. However, the effect of approximating α_m from Equation 3.20 was found to be small (at least for the soil strength parameters considered in this project).

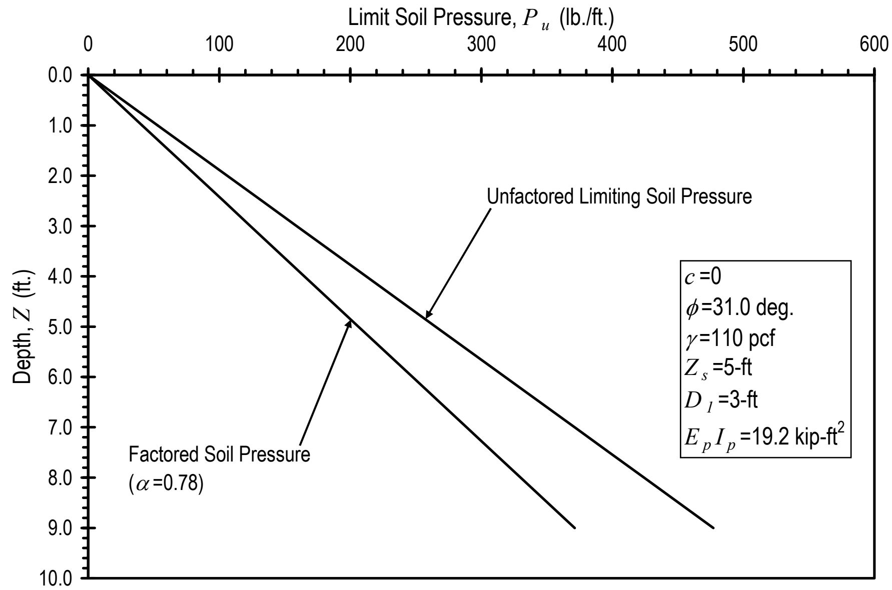


Figure 3.22 Factored pressure distribution considering moment capacity of reinforcing member.

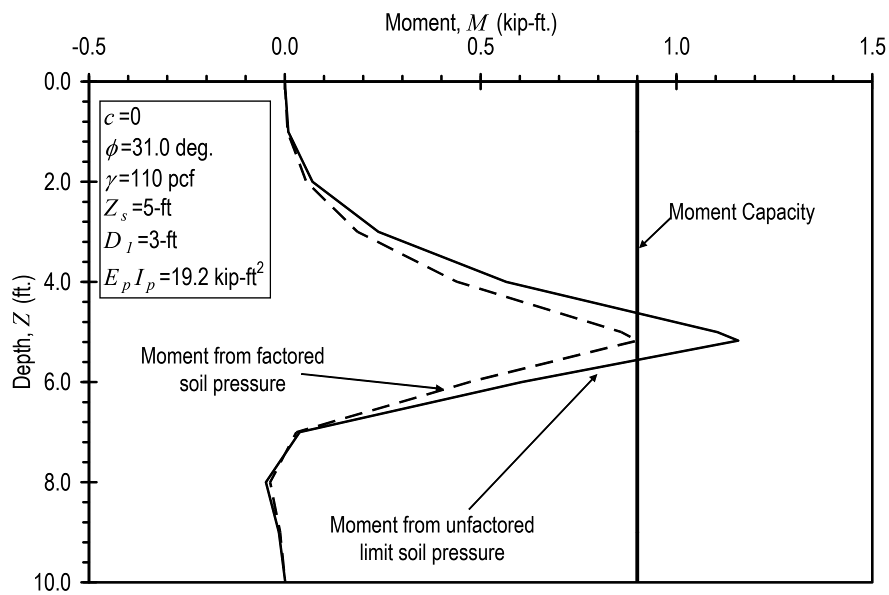


Figure 3.23 Factored and unfactored moment distribution diagrams.

The limiting resistance curves obtained considering both shear and moment for the conditions considered are compared in Figure 3.25. As shown, the limiting resistance computed considering the moment capacity of the reinforcing members is significantly lower than that for shear. Similar results were obtained for all cases analyzed during the project and moment capacity is expected to control for most types of members as long as they are solid circular or rectangular sections. If hollow sections, or other sections with relatively small cross-sectional areas are considered, shear capacity will be of more importance.

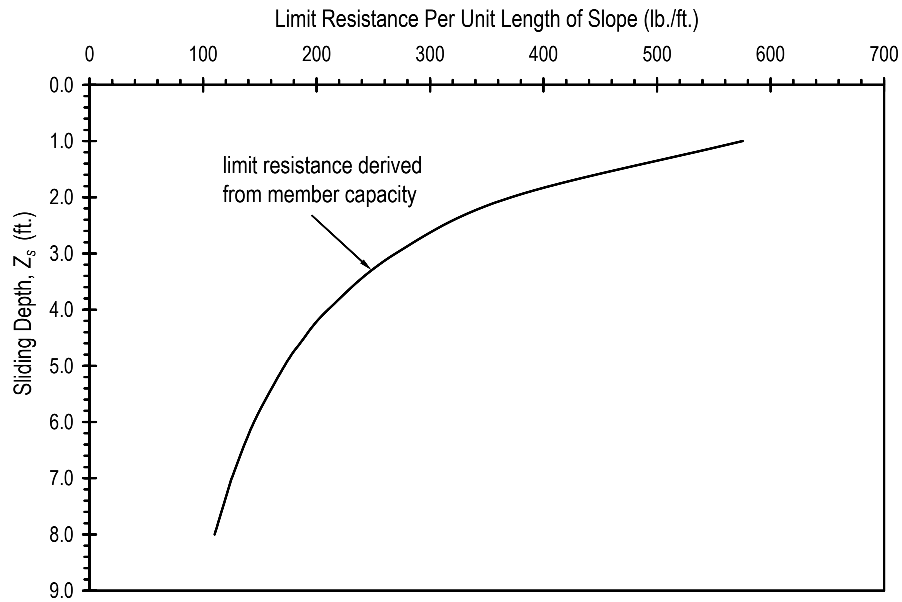


Figure 3.24 Limit resistance curve considering moment capacity of reinforcing member.

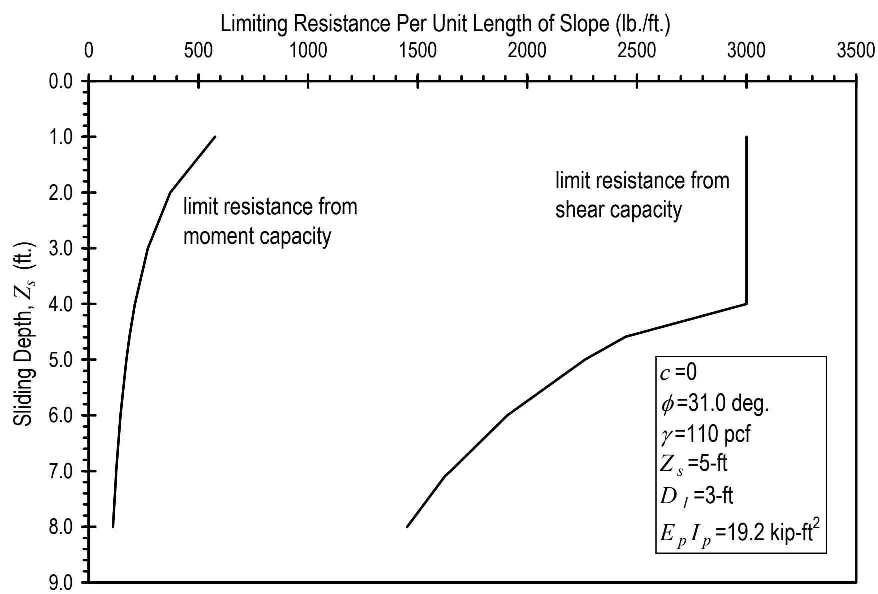


Figure 3.25 Comparison of limit resistance curves considering moment and shear of reinforcing member.

Once the limit resistance distributions are determined, a “composite” limit resistance curve that is suitable for “weak” recycled plastic members is obtained by taking the least of the three resistances at each sliding depth as shown in Figure 3.6 and 3.7. As shown in Figure 3.7, the limit resistance generally increases with sliding depth throughout the zone controlled by the soil resistance then decreases with sliding depth throughout the zone controlled by the reinforcing member capacity. The resistance then decreases to zero at the end of the member controlled by the anchorage lengths.

3.3. Calculation of Limiting Resistance for Inclined Reinforcing Members

The limiting soil pressure utilized in computing the limiting resistance distributions as described in Section 3.2 is based on the assumption of vertical reinforcing members in horizontal ground as shown in Figure 3.26. How the limiting soil pressure changes for vertical reinforcing members in a slope (Figure 3.27) or for inclined reinforcing members (Figure 3.28) is not well understood. No other methods appropriate for these conditions were found in the literature. As such, several approximations were made for using the Ito and Matsui limiting soil pressures for vertical members in sloping ground and for inclined members. Analyses were then performed to study the impact of the modification in calculating limiting soil pressure.

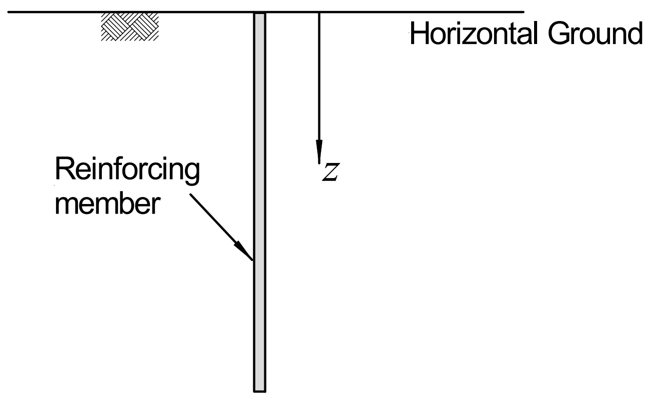


Figure 3.26 Reinforcing member and ground geometry assumed in Ito and Matsui method (vertical member in horizontal ground).

For vertically oriented reinforcing members in sloping ground, the limiting soil pressure was assumed to be identical to that computed using the Ito and Matsui method for vertical members in horizontal ground. While the influence of this approximation is not known at this time, it is believed to have negligible effect on the limiting resistance of reinforcing members since the effective overburden stress (γz) acting on a reinforcing member at a depth z is similar to that which would be computed for horizontal ground (Figure 3.27).

For inclined reinforcing members, the picture is much less clear. As shown in Figure 3.28, the effective overburden pressure at a point at distance z from the top of the reinforcing member is not given by γz, but rather is a function of the relative inclination of the slope and the reinforcing member. Two alternative methods for calculating the limiting resistance were therefore evaluated for use with inclined reinforcing members. The first alternative used was to simply ignore the influence of inclination and use the limiting resistance distribution for the reinforcing members as if they were placed vertically. The second alternative used was to compute the limiting soil pressure assuming that the effective overburden stress is given by the height of soil above a particular point on the reinforcing member as shown in Figure

3.28. For reinforcing members placed perpendicular to the face of the slope, the vertical overburden pressure for the second alternative is given by

$$\overline{\sigma_v} = \frac{\gamma z}{\cos\beta} \tag{3.21}$$

where γ is the unit weight of the soil, z is the distance from the ground surface to the point of interest measured along the reinforcing member, and β is the inclination of the slope.

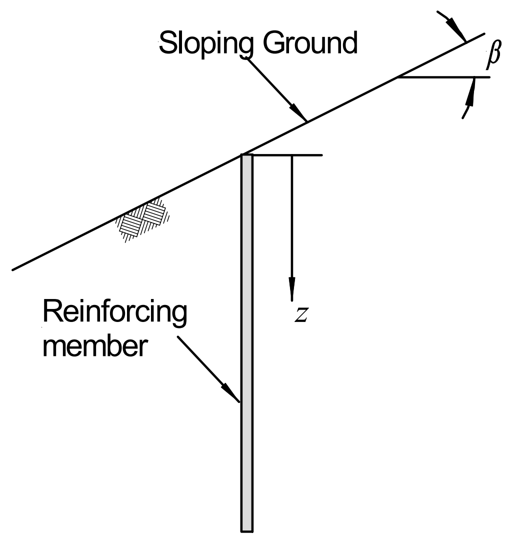


Figure 3.27 Vertical reinforcing member in sloping ground.

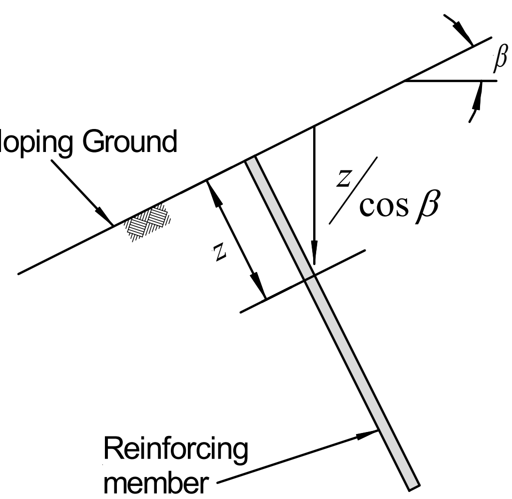


Figure 3.28 Inclined reinforcing member in sloping ground.

Figures 3.29 and 3.30 show limit resistance curves for "weak" and "strong" reinforcing members respectively placed perpendicular to the face of the slope using the two alternative methods for computing the limit soil pressure. As shown in the figures, the differences in the computed limit resistance curves are small and therefore are not likely to significantly influence the computed factors of safety. Additional calculations comparing factors of safety computed using the two alternative methods described above suggest a similar conclusion (Liew, 2000).

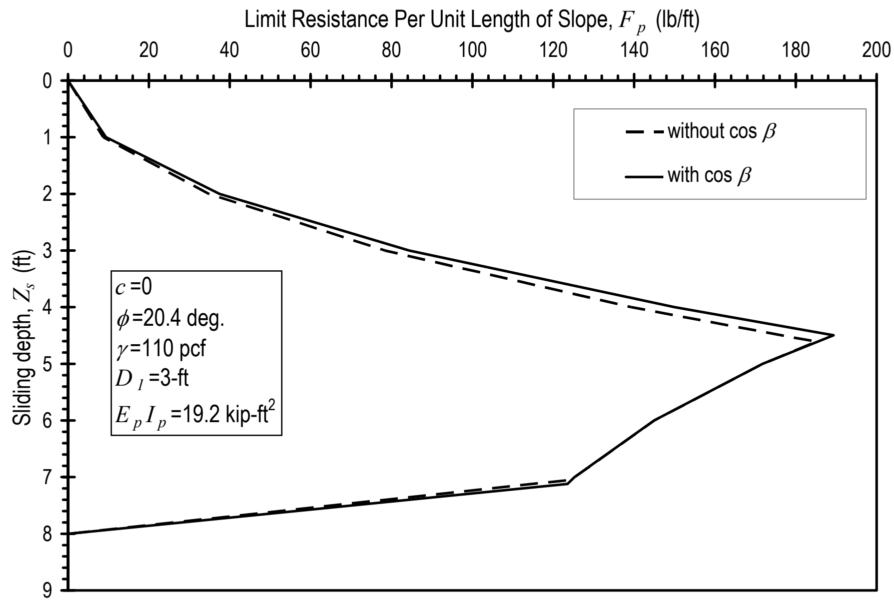


Figure 3.29 Limit resistance curves for "weak" reinforcing members placed perpendicular to the face of the slope computed using two alternative methods.

3.4. Calculation of Factor of Safety

Once the overall limit lateral resistance distribution is developed for individual reinforcing members, the mechanics of stability analysis for slopes reinforced with structural members are relatively straightforward and well established. The commercial slope stability analysis software, UTEXAS4, was used to perform all of the stability analyses presented in this report. The program has the ability to search for the most critical sliding surface and the minimum factor of safety with or without reinforcing elements.

Although the process of calculating the factor of safety using UTEXAS4 is straightforward once the overall limiting lateral resistance curve is developed, the searches for the most critical sliding surface (the surface giving the minimum factor of safety for a particular set of slope conditions) proved to be difficult as numerous "local minima" exist as sliding surfaces passing through different zones of the reinforcement are considered. To ensure that the overall most critical sliding surface was found for all analyses, a rigorous search procedure was developed. The procedure consisted of:

(1) selecting several starting centers of circles for the searches (6-10) around the toe, face and crest of slope;

(2) for each center of circle chosen, trying a radius crossing reinforcement with the sliding surfaces less than 4 ft (1.2-m) and 8 ft (2.4-m) below the face of the slope;

(3) from the most critical circle found from step (2), repeat searches by starting the search using circles with the same center and varying the radius within

approximately 2-ft (0.6-m) of the radius of that previous critical circle. The critical circle was determined to a precision of 0.5 ft (0.15-m). This procedure proved to be effective for finding the minimum factor of safety within a tolerance of 0.001.

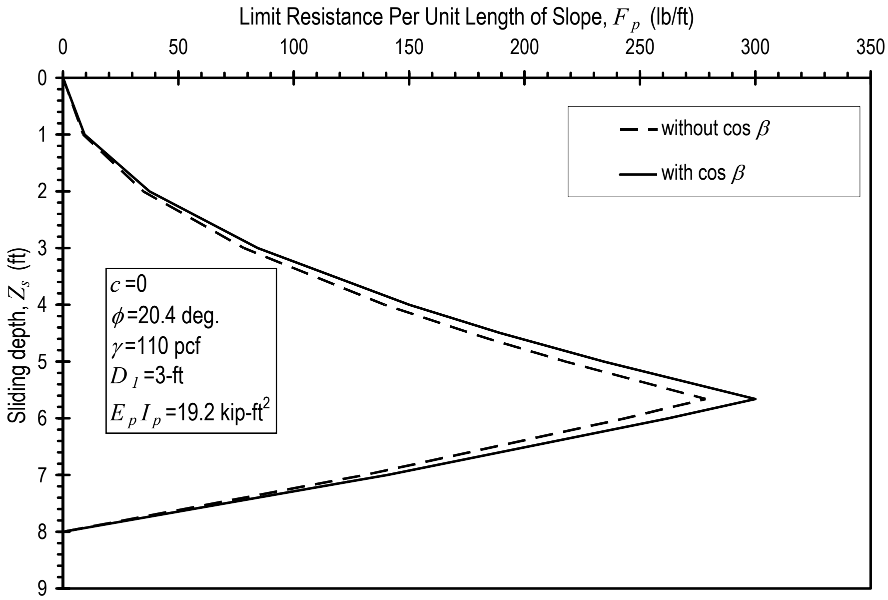


Figure 3.30 Limit resistance curves for “strong” reinforcing members placed perpendicular to the face of the slope computed using two alternative methods.

3.5. Summary

A procedure for computing factors of safety for slopes reinforced with distributed arrays of reinforcement was presented. The procedure consists of first computing the limit lateral resistance distributions of individual reinforcing members and then inputting the resistance distribution into commercial slope stability analysis software to compute the factor of safety. The method for developing the limit lateral resistance distributions uses a limit state design approach wherein failure of soil surrounding the reinforcing members, structural failure of the reinforcing members and failure due to insufficient anchorage length beyond the sliding surface are considered separately. The limit resistance distributions computed for these three limit states are then combined to form a composite limit resistance distribution that can be utilized with commercial slope stability analysis programs.

Chapter 4. Engineering Properties of Recycled Plastic Members

The engineering properties of recycled plastic reinforcing members are of paramount importance because of the potential for structural failure of the members due to the loads imparted on the members during field installation or imposed by the moving soil following installation. An extensive testing program has therefore been undertaken to develop a database of the engineering properties of recycled plastic members. The program has included: (1) determining the basic engineering and material properties of recycled plastic members; (2) determining how these properties change when the material is subjected to potentially detrimental environments; and (3) determining the potential variability of these properties within one product and among various products and manufacturers. Much of the basic evaluation of member properties, including evaluation of the effects of environmental conditions, was performed during Phase I and presented in the final report for Phase I (Loehr et al. 2000). Additional testing to evaluate the potential variability of recycled plastic products and address several other issues was performed during Phase II, with supplemental funding provided by the Recycled Materials Resource Center (RMRC). The expanded testing program undertaken during Phase II is described in this chapter along with a summary of the results obtained from these efforts. Additional details regarding these efforts can be found in Chen (2003) and Bowders et al. (2003).

4.1. Recycled Plastic Members

Laboratory tests were performed on specimens from three manufacturers denoted Manufacturer A, Manufacturer B, and Manufacturer C. Members were divided into thirteen different batches according to the type of member, the manufacturer, and the condition of the member as shown in Table 4.1. All of the members were nominally 3.5-in. x 3.5-in. (90-mm x 90-mm) in cross-section by 8-ft (2.4-m) in length. The principal constituent and manufacturing processes for each manufacturer varied as did the measured unit weights, which ranged from 52- to 68-pcf (8- to 11-kN/m^3).

Manufacturer A provided members manufactured in seven different batches, denoted batches A1 through A6 and A10, over a period of three years. Members in batches A1 through A4 were compression-molded products while members from batches A5, A6 and A10 were extruded products. The constituent formula among the first five batches (A1-A5) was similar with approximately 60 percent low-density polyethylene (LDPE) and 40 percent filler material (primarily sawdust). Batches A6 and A10 were produced using high-density polyethylene (HDPE) instead of LDPE and had a lower proportion of filler materials. Specimens from Batches A11, A12, and A13 were taken from the portion of members that remained above ground after installation. These members were all manufactured at the same time and with the same constituent formula as Batch A10; however, these specimens are considered "disturbed" because they were subjected to driving stresses similar to other members installed in the field. Batches A11 and A12 were taken from members installed at the I70-Emma test site in January 2003 while Batch A13 was composed of members installed at the US54-Fulton test site in January 2003.

Two additional manufacturers (Manufacturers B and C) provided specimens of unreinforced members composed of HDPE with negligible filler materials. These specimens are denoted as Batches B7 and C9. Manufacturer B also provided specimens composed of

HDPE reinforced with cut-strand fiberglass reinforcement. This batch of specimens is denoted Batch B8. Specimens from batches A1 through A6, A10, B7, B8, and C9 were manufactured at company facilities and shipped to the University of Missouri-Geotechnical Laboratories for testing or to the contractor for installation at the field test sites and are considered "virgin" materials (undisturbed).

Table 4.1 Summary of recycled plastic members used in the laboratory testing program.

Specimen Batch	Principal Constituent	Mftg. Process	Condition	Depth (in)	Width (in)	Length[1] (in)	Unit weight (lb/ft^3)
A1	LDPE	Compression	virgin	3.6	3.6	7.0	61.2
A2	LDPE	Compression	virgin	3.5	3.5	6.9	63.4
A3	LDPE	Compression	virgin	3.6	3.6	7.1	64.5
A4	LDPE	Compression	virgin	3.6	3.4	7.0	64.6
A5	LDPE	Extruded	virgin	3.4	3.4	7.1	58.9
A6	HDPE	Extruded	virgin	3.4	3.4	7.0	60.9
A10	HDPE	Extruded	virgin	3.5	3.5	7.0	67.6
A11	HDPE	Extruded	disturbed	3.5	3.5	7.0	68.3
A12	HDPE	Extruded	disturbed	3.5	3.5	7.0	68.5
A13	HDPE	Extruded	disturbed	3.5	3.5	7.0	66.8
B7	HDPE	Extruded	virgin	3.4	3.4	6.9	52.9
B8	HDPE + Fiberglass	Extruded	virgin	3.4	3.4	6.9	51.9
C9	HDPE	Extruded	virgin	3.5	3.5	7.0	67.9

[1] for uniaxial compression tests.

4.2. Laboratory Testing Program

Laboratory tests performed to evaluate the engineering properties of the recycled plastic members included uniaxial compression tests, four-point flexure tests, compressive creep tests, and flexural creep tests. When available, standard ASTM test methods for recycled plastic lumber products (Table 4.2) were followed; however, some specific steps were modified to produce results more appropriate for the application of recycled plastic lumber for stabilization of earthen slopes. The principal change in the test procedures was a reduction in strain or deformation rate during loading as described in more detail in Chen (2003) and Bowders et al. (2003).

4.3. Engineering Properties of Recycled Plastic Members

A summary of results from the laboratory testing program is presented in this section. Properties reported include uniaxial compressive strength, modulus of elasticity in compression, flexural strength, and modulus of elasticity in flexure. Results of accelerated creep testing are also summarized. More detailed discussions of the results of the testing program can be found in Chen (2003) and Bowders et al. (2003).

Table 4.2 ASTM Standard Test Methods for Plastic Lumber.

ASTM No. & Title	Test Method	Comments
D6108 Standard Test Method for Compressive Properties of Plastic Lumber and Shapes	Uniaxial Compression Test	• Specimens: length = 2 x minimum width. • Compressive stress = compressive load divided by minimum or effective original cross-sectional area. • Choose 3% strain as compressive strength if no clear yield point. • Strain rate = 0.03 ± 0.003 in/in/min (mm/mm/min) and testing time ~ 1 to 5 min. • Secant Modulus @ 1% strain.
D6109 Standard Test Method for Flexural Properties of Unreinforced and Reinforced Plastic Lumber	Four-point Flexure Test	• Specimens: support span (length) divided by minimum width = 16 (nominally). • Calculated rate of crosshead motion by equation listed in standard. • Flexural strength = max. stress at rupture. • Secant modulus of elasticity from equation provided.
D6112 Standard Test Methods for Compressive and Flexural Creep and Creep-Ruptured of Plastic Lumber and Shapes	Compressive Creep and Flexural Creep	• Uniaxial type of loading for compressive creep. • Plot successive creep modulus versus time at various stresses for linear viscoelasticity materials. • Four-point flexure testing set-up for flexural creep. • Approximate time schedule for compressive or flexural creep tests: 1, 6, 12, and 30 min; 1, 2, 5, 20, 100, 200, 500, 700, and 1000 hours. • Able to predict the creep modulus and strength of material under long-term loads from testing data.

4.3.1. Compression Test Results

The average and standard deviation of the compressive strengths measured for each batch of specimens using a nominal strain rate equal to 0.006-in/in/min (0.006 mm/mm/min) are summarized in Table 4.3. Overall, measured compressive strengths ranged from 1600- to 3000-psi (11- to 21-MPa) when calculated using the original cross-sectional area of the member. The compression-molded products tended to have the highest uniaxial compressive strengths while the extruded products tended to have somewhat lower compressive strengths. Batches A5 and A6 produced the lowest mean compressive strengths. However, these batches were produced by Manufacturer A during startup and testing of new manufacturing equipment. Subsequent batches produced using the same equipment had compressive strengths that were similar to other extruded products. Based on this information, nominal strengths for compression molded products are estimated to be on the order of 2800-psi while the strength of extruded products is nominally 2000- to 2500-psi.

Mean values and standard deviations of the secant modulus of elasticity, *E*, determined from the uniaxial compression tests at one and five percent strain are summarized in Table 4.4. At one percent strain, the secant modulus ranged from 84- to 184-ksi (580- to 1269-MPa). At five percent strain, values ranged from 32- to 57-ksi (220- to 393-MPa). Compression-molded products tended to have higher average moduli. Extruded products from batches A5, A6, B7, and C9 had the lowest moduli determined at 1 percent strain. The remaining extruded products had intermediate moduli.

Table 4.3 Uniaxial compressive strengths determined for recycled plastic members.

Specimen Batch	# Specimens Tested	Nominal Strain Rate (in/in/min)	Uniaxial Compressive Strength (psi)			
			Initial Area[1]		Actual Area[2]	
			Mean	Std. Dev.	Mean	Std. Dev.
A1	10	NA	2784	128	--[3]	--
A2	7	0.005	2948	117	--	--
A3	6	0.005	2824	88	--	--
A4	6	0.005	2621	295	2486	271
A5	6	0.007	1634	200	1578	189
A6	14	0.007	1602	105	1521	102
A10	15	0.006	2219	154	2152	136
A11	15	0.006	2301	139	2217	140
A12	8	0.007	2085	84	1931	199
A13	15	0.007	2380	330	2310	318
B 7	15	0.007	2080	69	2331	134
B 8	15	0.006	2500	191	2505	195
C 9	15	0.007	2315	209	2556	322

[1] stress calculated using initial cross-sectional area (A_0)
[2] stress calculated using corrected cross-sectional area (A_c)
[3] Data not available

4.3.2. Strain Rate Effects on Strength

The strain rate used for testing plastic products has particular significance in developing a suitable specification for recycled plastics in the slope stabilization application. Several ASTM standards have recently been developed specifically for testing plastic lumber products as summarized in Table 4.2. These standards dictate use of strain rates that are approximately 1.5 times greater than the highest strain rate used in the present testing. While the value of standardized test procedures is acknowledged, current standards were developed with typical building applications in mind. The loading rates specified in these standards are therefore very high. In the slope stabilization application, members are called upon to resist sustained bending loads over time, which may cycle from negligible load to the limit loads of the members as load is transferred from the moving soil in response to environmental conditions in the slope. In this application, the loading rate is likely to be very slow – on the order of weeks or months (seasonal). The evaluation program therefore included tests performed at a range of loading rates to establish relationships between the properties of interest (e.g. strength and stiffness) and loading rate.

Figure 4.1 shows a summary of normalized compressive strengths determined from the testing program plotted as a function of the strain rate used to measure the strength. In the figure, the compressive strengths are normalized with respect to the strength that would be measured at the rate specified in ASTM D6108 (0.03-in/in/min). The figure clearly shows that the strength is highly dependent on the strain rate. Furthermore, the figure shows that the strain rate effects for different products can be markedly different as evidenced by the upper and lower bound curves shown in Figure 4.1. If the field strain rate is taken to be 0.00003 in/in/min (approx. three orders of magnitude slower than the ASTM rate), the data indicate that field strengths may be from 10 to 70 percent of the strength measured at the ASTM strain rate. Thus, strengths reported by manufacturers must be accompanied by the strain rate used in their tests, so that designers can determine appropriate design strengths. More information on the development of the data shown in Figure 4.1 is given in Chen (2003) and Bowders et al. (2003).

Table 4.4 Secant modulus of elasticity determined for recycled plastic members.

			Secant Modulus of Elasticity (ksi)							
			Initial Area[1]				Corrected Area[2]			
			$E_{1\%}$		$E_{5\%}$		$E_{1\%}$		$E_{5\%}$	
Specimen Batch	# Specimens Tested	Nominal Strain Rate (in/in/min)	Mean	Std. Dev.	Mean	Std. Dev.	Mean	Std. Dev.	Mean	Std. Dev.
A1	10	NA	134	8	57	4	--[3]	--	--	--
A2	7	0.005	184	9	55	3	--	--	--	--
A3	6	0.005	164	29	57	3	--	--	--	--
A4	6	0.005	186	20	52	4	185	20	49	4
A5	6	0.007	84	16	33	4	84	16	31	3
A6	14	0.007	93	8	32	2	92	8	30	2
A10	15	0.006	114	12	45	3	113	12	43	3
A11	15	0.006	119	11	47	3	119	11	45	3
A12	8	0.007	108	11	40	4	107	11	38	4
A13	15	0.007	110	21	48	6	110	21	45	6
B7	15	0.007	87	10	42	2	85	11	39	3
B8	15	0.006	138	27	49	4	136	26	47	4
C9	15	0.007	87	12	46	4	86	12	45	4

[1] stress calculated using initial cross-sectional area (A_0)
[2] stress calculated using corrected cross-sectional area (A_c)
[3] Data not available

4.3.3. Four-Point Flexural Test Results

Results of the four-point flexure tests are summarized in Table 4.5. Since the number of tests on batches A11 and A12 were limited, no standard deviation is reported. In the tests, extruded members showed continually increasing stress with increasing deflection/strain without experiencing rupture of the member while compression molded members ruptured at approximately two percent strain. The flexural strengths for the different products was therefore taken to be the flexural stress at center strains of two percent, or the stress at rupture for members that failed at center strains of less than two percent, to ensure that consistent strengths were established for all specimens. Measured flexural strengths ranged from 1300- to 3600-psi (9- to 25-MPa) for a nominal deformation rate of 0.2-in/min (5.1-mm/min). The key finding from these tests is that there is significant variability, by a factor of approximately 2.8, in the flexural strength among the products tested.

Average values of the secant flexural modulus for each batch of specimens are also listed in Table 4.5. In general, the flexural moduli varied from approximately 90- to 250-ksi

(621- to 1724-MPa) at one percent strain. These values are similar to the values determined from uniaxial compression tests with the exception of batch B8.

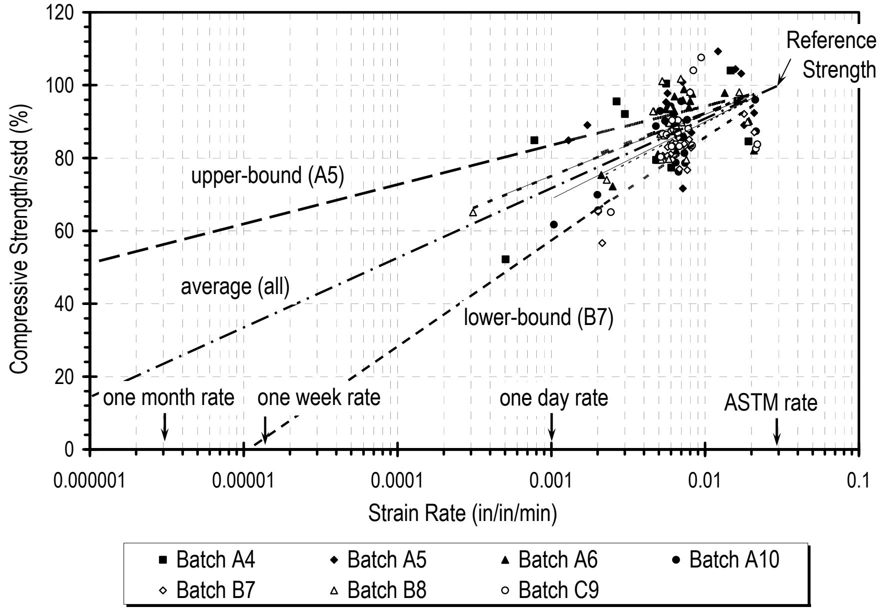


Figure 4.1 Normalized uniaxial compressive strength versus strain rate for recycled plastic members from different batches.

Table 4.5 Summary of results from four-point flexure tests for recycled plastic members.

Specimen Batch	# Specimens Tested	Nom. Def. Rate (in/min)	Flexural Strength[1] (psi) Mean	Flexural Strength[1] (psi) Std. Dev.	Secant Flexural Modulus (ksi) $E_{1\%}$ Mean	$E_{1\%}$ Std. Dev.	$E_{2\%}$ Mean	$E_{2\%}$ Std. Dev.
A1	13	--[2]	1574	342	103	8	88[3]	--
A4	3	0.17	2543	260	213	13	--	--
A5	5	0.23	1542	188	98	14	73	2
A6	7	0.14	1360	118	95	12	68	6
A10	6	0.18	1596	137	123	22	76	10
A11	1	0.19	1679	--	135	--	81	--
A12	1	0.19	1448	--	115	--	71	--
B7	6	0.17	1505	112	90	7	69	4
B8	6	0.17	3589	358	243	24	179	13
C9	7	0.16	1696	39	107	4	83	2

[1] all results based on stress at 2% center strain or center strain at rupture if less than two percent
[2] data not available
[3] result of 2 specimens, others ruptured prior to reaching two percent center strain
Conversion: 1-MPa=145-psi, 1-ksi=6.9-MPa

Results from batches A4 and B8 indicate significantly higher flexural strength and stiffness than the other batches. This is likely a result of being compression-molded or reinforced as compared to being unreinforced extruded products. Breslin et al. (1998)

concluded that the use of glass and wood fiber additives significantly improves the modulus of elasticity for plastic lumber. Batch A10 (virgin specimens), and Batches A11 and A12 (disturbed specimens) have similar flexural strength and flexural moduli, which suggests that negligible change in properties occurs due to installation using current methods. The data also show that the flexural moduli at two percent center strain were consistently lower than those determined at one percent center strain, indicating the members tended to soften with increasing strain.

4.3.4. Creep Test Results

Flexural and compressive creep tests were also performed on selected recycled plastic members. The flexural creep tests revealed the recycled plastics to be creep sensitive with the creep rate highly dependent on the temperature and stress level in the members. However, Arrhenius modeling performed using the data obtained from this project indicates that, under current field stress levels at the I70-Emma test site, the members would not reach creep failure for 45 to 2000 years. Extensive details of the creep testing, and results of the Arrhenius modeling are presented in Chen (2003) and Bowders et al. (2003).

4.4. Summary

Results obtained from additional laboratory testing of recycled plastic members during Phase II have been summarized in this chapter. These results indicate that the compressive strengths of recycled plastic members range from 1600- to 3000-psi (11- to 21-MPa) with extruded products having strengths about 20 percent lower than compression-molded products. Values for the secant modulus of elasticity from uniaxial compression tests ranged from 80- to 190-ksi (552- to 1310-MPa) at one percent strain. Compression-molded products had the highest moduli of all products. Fiberglass-reinforced products were about 60 percent stiffer than unreinforced products. Flexural strengths ranged from 1300- to 3600-psi (9- to 25-MPa), but there was significant variability. The flexural moduli varied from 90- to 250-ksi (621- to 1724-MPa) at one percent strain. Although recycled plastic members are creep sensitive, Arrhenius modeling indicates that, at field temperature and stress levels, creep failure is not expected for between 45 and 2000 years.

Chapter 5. I435-Kansas City Sites

Two of the selected field test sites are located in southern Kansas City Missouri in close proximity to one another along Interstate 435 near the Missouri-Kansas border. The first site is located at the intersection of I435 and Wornall Road; the second is located due east at the intersection of I435 and Holmes Road. This chapter contains descriptions of the two stabilization sites, the project activities undertaken to construct the sites, the instrumentation used to monitor the sites, and a summary of the field performance of each slope to date.

5.1. Site Characteristics

A map showing the locations of the I435-Kansas City sites is shown in Figure 5.1. The I435-Wornall Road test site is located in the northeast quadrant of the intersection of I435 and Wornall Road between I435 and the westbound exit ramp. The I435-Holmes Road test site is located approximately one mile east of the I435-Wornall Road site. The slope lies in the southeast quadrant of the intersection of I435 and Holmes Road between I435 and the eastbound entrance ramp. Both slopes are bridge approach embankments that serve to support I435 as it passes over Wornall and Holmes roads, respectively. A third slope located in the southwest quadrant of the I435-Wornall Road intersection is being used as a control section for both of these sites.

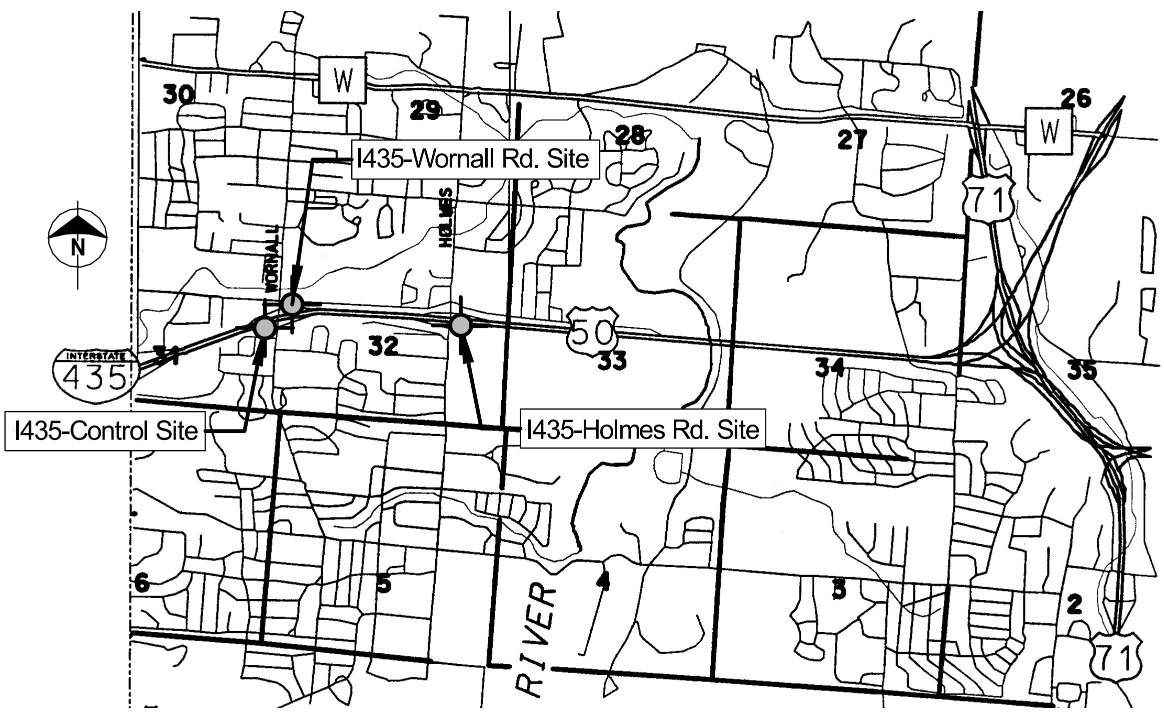


Figure 5.1 Map of southern Kansas City Missouri showing locations of I435-Kansas City test sites.

5.1.1. I435-Wornall Road Site

The I435-Wornall Road embankment is a zoned-fill embankment consisting of a 3- to 5-ft (0.9- to 1.5-m) thick surficial layer of mixed lean to fat clay with soft to medium consistency, overlying stiffer compacted clay shale. The embankment is approximately 32-ft (9.6-m) high with side slopes of 2.2H:1V (horizontal:vertical). The embankment had experienced surficial slides along the interface between the upper clay and the lower compacted clay shale on at least two occasions prior to being selected for stabilization as part of this project. Figure 5.2 shows a photograph of the site following the most recent slide event, which affected approximately 125-ft (38-m) of the embankment (measured parallel to I435). Prior to the most recent slide, extensive ornamental vegetation had been placed on the slope along with 4- to 6-in (100- to 150-mm) of gardening mulch as a part of a neighborhood beautification project. The most recent slide took out a large amount of this vegetation and caused much of the gardening mulch to become mixed with the surficial soils.

Figure 5.2 Photograph of most recent slide at I435-Wornall Road test site, June 20, 2001.

Boring and sampling at the I435-Wornall Road site was performed by MoDOT drilling crews during the period June 25-27, 2001. A total of seven hollow-stem auger borings were made in the slide area to depths varying from 10- to 30-ft (3- to 9-m). A plan view of the site showing boring locations is provided in Appendix A along with logs of all borings. In all but one boring, continuous 3-in (7.6-cm) diameter Shelby tube samples were taken and extruded in the field for visual description and field testing. Samples were then wrapped in aluminum foil and sealed with paraffin for subsequent transportation to the Geotechnical Engineering laboratories at the University of Missouri-Columbia. Obtaining good quality samples at shallow depths within the slide area proved difficult because of the presence of the gardening mulch that had become mixed with the surficial soils. In one boring, Standard Penetration Tests (SPT) were performed at 1.5-ft (0.5-m) intervals until refusal. SPT N_{60}-values reported for the surficial layer ranged from 0 (weight of hammer) at the surface to 2 at a depth of 4-ft (1.2-m) while values at greater depth ranged from 6 to 16. Auger refusal was generally encountered at a horizontally bedded limestone layer located at depths ranging from 12-ft (3.7-m) near the toe of the slope to 32-ft (9.8-m) at the crest. Groundwater was not observed in any of the boreholes during boring and sampling; however, a groundwater condition was evident from the presence of water on the slope face within the lower third of the slope.

Laboratory tests performed on samples obtained from the I435-Wornall Road test site included natural moisture content tests, Atterberg limits, and triaxial tests. Results of these tests indicate soils from the surficial layer had liquid limits (LL) ranging from 38 to 51 and plasticity indices (PI) from 16 to 34. The compacted shale present below the surficial layer had LL ranging from 29 to 76 and PI from 12 to 51.

Consolidated-undrained ($\overline{\text{CU}}$, $\overline{R}$) type triaxial compression tests with pore water pressure measurements were performed on a total of 10 specimens from the I435-Wornall Road site. Stress paths determined from these tests are plotted in Figure 5.3 along with failure envelopes established for surficial (< 4-ft) and deeper (> 4-ft) soils. Mohr-Coulomb strength parameters for these envelopes are summarized in Table 5.1. For the surficial soils (Fig. 5.3a), all tests indicated a consistent effective stress failure envelope with $\bar{c} = 0$ and $\bar{\phi} = 27$ degrees. For deeper soils, three different effective stress failure envelopes were established: a "lower bound" failure envelope, an "upper bound" failure envelope, and an "alternative" failure envelope as shown in Figure 5.3b. For the deeper soils, $\bar{c}$ was found to vary between 0- and 120-psf (5.7 kPa) and $\bar{\phi}$ was found to vary between 23 and 31 degrees.

Table 5.1 Summary of Mohr-Coulomb effective stress strength parameters from triaxial compression tests on specimens from the I435-Wornall Road test site.

			upper bound		lower bound		"alternative"	
Stratum	Depths	Sample Numbers	$\bar{c}$ (psf)	$\bar{\phi}$ (°)	$\bar{c}$ (psf)	$\bar{\phi}$ (°)	$\bar{c}$ (psf)	$\bar{\phi}$ (°)
Surficial clay	< 4.0-ft	38A, 38B 106, 152	0	27	--	--	--	--
Deeper clay	> 4.0-ft	60, 64, 108, 111, 142A, 142B	0	31	0	26	120	23

5.1.2. I435-Holmes Road Site

The I435-Holmes Road site embankment also consists of a surficial layer of mixed lean to fat clay overlying compacted clay shale. The surficial layer varied in thickness between 3- and 6-ft (0.9- to 1.8-m). The site had experienced at least one failure prior to being selected for stabilization as a part of this project. Figure 5.4 shows a photograph of the slope following this slide event. At the location of the slide, the embankment is approximately 15-ft (4.6-m) high with a slope varying from 2.2H:1V near the crest to 2.6H:1V near the toe of the slope.

No site investigation was performed at the I435-Holmes Road site prior to stabilization. However, one boring made during installation of an inclinometer for the test site was logged on July 11, 2002. Logs from this boring, located near the center of the slide area, indicate that the surficial lean clay material at this location was approximately 4.5-ft (1.4-m) thick and of medium stiff consistency. This layer is underlain by approximately 1-ft (0.3-m) of medium stiff, compacted clay shale, which in turn is underlain by much stiffer clay shale. SPT N_{60}-values determined at depths between 5.3- and 7.8-ft exceeded 100 blows per foot.

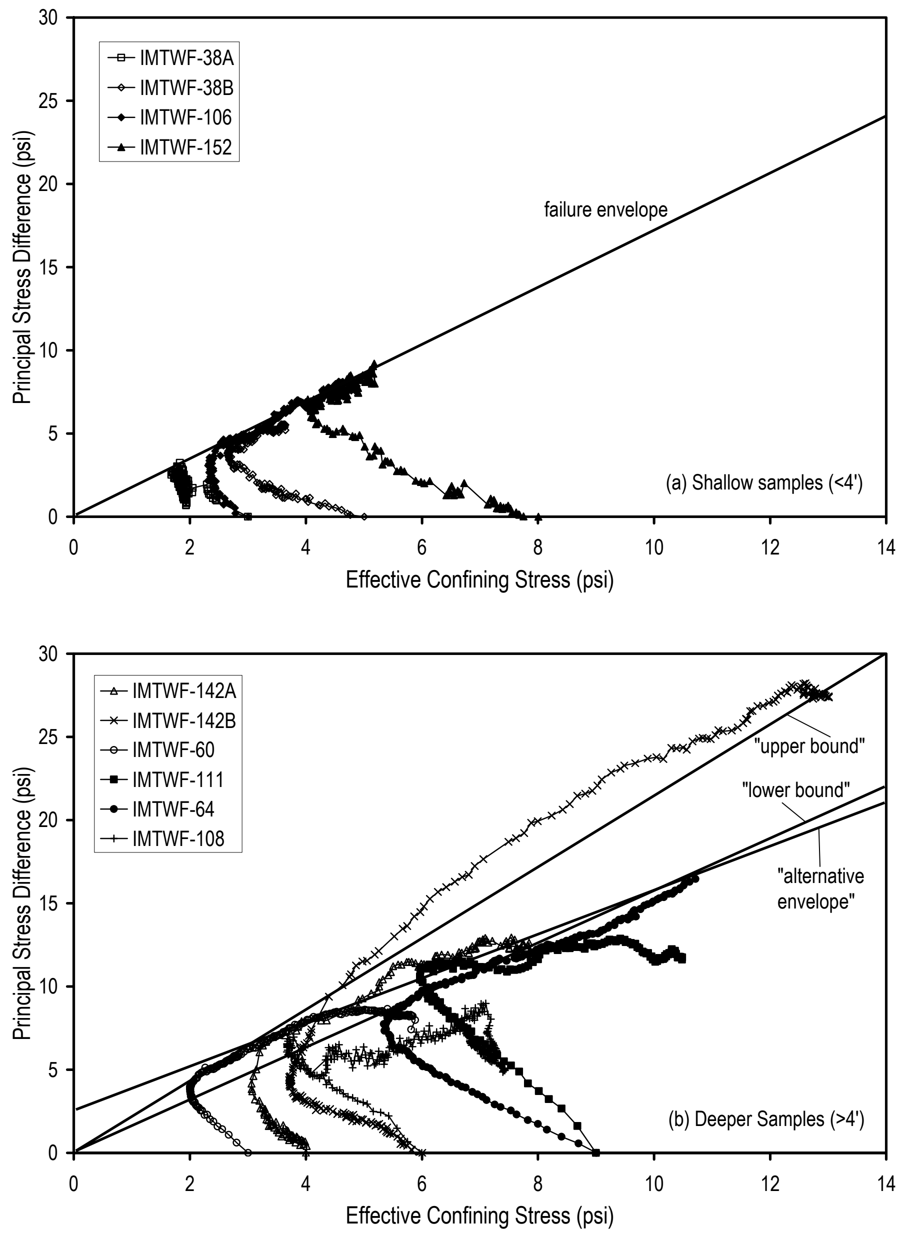


Figure 5.3 Summary of triaxial test results for specimens from I435-Wornall Road site: (a) shallow samples and (b) deeper samples.

Figure 5.4 Photograph of the I435-Holmes Road site prior to stabilization.

5.1.3. I435 Control Slide

A third slide area, located in the southwest quadrant of the intersection of I435 and Wornall Road and shown in Figure 5.5, was selected for use as a control section for both the I435-Wornall and I435-Holmes test sites. One boring was made at this site on July 10, 2002 during installation of instrumentation for the slope. This boring revealed that the stratigraphy of the control slide was similar to that observed for the I435-Wornall Road and I435-Holmes Road slides with 5- to 7-ft of mixed lean to fat clay overlying hard compacted clay shale.

5.2. Design of Stabilization Schemes

An extensive series of stability analyses were performed to establish the stabilization schemes to be utilized at the respective sites. All analyses were performed using the commercial slope stability software UTEXAS4 (Wright, 2001), which utilizes the limit equilibrium approach. Stability analyses for alternative reinforcement scenarios were performed in accordance with procedures described in Chapter 3. The following sections describe the stability analyses performed and the stabilization scheme selected for each site.

5.2.1. Stabilization Scheme for I435-Wornall Road Site

The general design cross-section used for the I435-Wornall Road site is shown in Figure 5.6. The ground surface profile was determined from a survey performed for MoDOT following the most recent slide event. The subsurface geometry was assumed to consist of a relatively thin surficial layer overlying compacted clay shale based on results of boring and sampling at the site. The thickness of the surficial layer was varied between 3-ft (0.9-m) and 5-ft (1.5-m) for different analyses.

Because the landscaping mulch became intermixed with the surficial soils during the slide, it was difficult to obtain high quality specimens of the surficial soils for testing. The strength parameters determined for this material were therefore deemed questionable. As a result of this, and the fact that the pore pressure conditions leading to the failure were unknown, a series of back-analyses was performed to establish several plausible sets of conditions that could have led to the failure. All analyses were effective stress stability analyses assuming fully drained, steady state seepage conditions.

Figure 5.5 Photograph of the I435 Control Slide prior to being regraded for use as a control section.

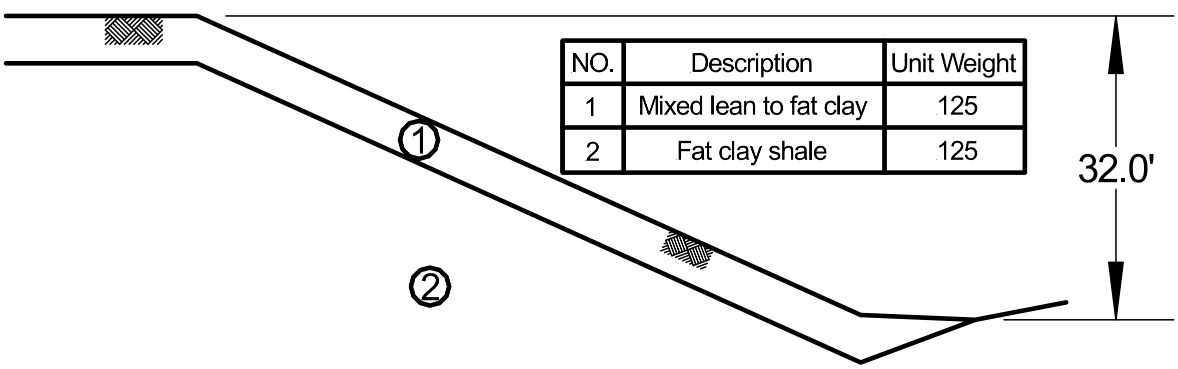


Figure 5.6 Design cross-section for I435-Wornall Road slope.

For the back-analyses, the strength parameters for the compacted clay shale material were taken to be $\bar{c} = 30$-psf and $\bar{\phi} = 27$ degrees based on laboratory tests performed on specimens from near the sliding surface. Several different pore pressure conditions were considered including a case where the pore pressures were assumed to be zero, a case where the pore pressures for both strata were defined by a piezometric surface lying at the ground surface, and a case with "perched" water conditions where pore pressures within the lower stratum were assumed to be zero and pore pressures within the upper stratum were defined by a piezometric surface coincident with the ground surface. For each of these pore pressure conditions and surficial layers of varying thickness, the value of $\bar{c}$ producing a factor of safety of 1.0 was back-calculated assuming that $\bar{\phi}$ for the surficial soil was either 10 or 20 degrees. For several additional cases, $\bar{c}$ was alternatively assumed to be zero and the value of $\bar{\phi}$ was back-calculated to give a factor of safety of 1.0. The results of all back-analyses were then evaluated with respect to whether they produced a critical sliding surface that was reasonably similar to the observed sliding surface. Conditions that did not produce a reasonable sliding surface were eliminated from further consideration. The conditions that did result in a reasonable critical sliding surface were considered plausible conditions leading to the failure. These conditions are summarized in Table 5.2.

Table 5.2 Summary of plausible conditions leading to the failure at the I435-Wornall Road site.

Stability Case	Thickness of Upper Stratum (ft)	Pore Pressure Condition	Assumed Strength Parameter	Back-calculated Strength Parameter
A	3.0	u=0	$\bar{\phi}$ =10°	$\bar{c}$ =79 psf
B	3.0-5.0	piezo. line – upper stratum	$\bar{\phi}$ =10°	$\bar{c}$ =99 psf
C	3.0-5.0	u=0	$\bar{\phi}$ =20°	$\bar{c}$ =21.5 psf
D	3.0	piezo. line – upper stratum	$\bar{\phi}$ =20°	$\bar{c}$ =64.5 psf
E	5.0	piezo. line – upper stratum	$\bar{\phi}$ =20°	$\bar{c}$ =100 psf
F	3.0-5.0	piezo. line – upper stratum	$\bar{c}$ =0	$\bar{\phi}$ =47°

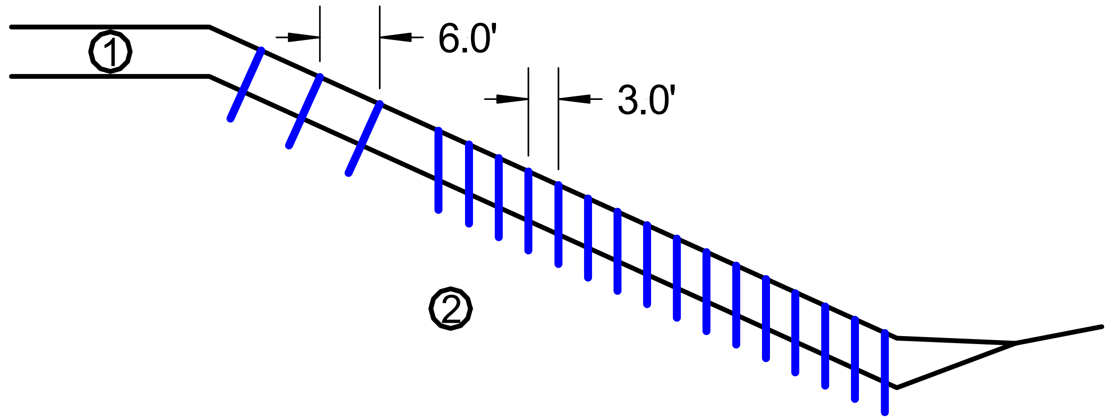
[1] thickness of upper stratum varies from 3-ft (0.9-m) at crest of slope to 5-ft (1.5-m) at toe

Once the plausible conditions that could have led to the failure were established, analyses were performed to estimate the factors of safety of the slope under a series of different reinforcement scenarios. Factors of safety computed for each of these reinforcement scenarios are summarized in Table 5.3 for each plausible stability case. Based on the results of these analyses, the reinforcement scheme shown in Figure 5.7 was selected for stabilization of the slope. The selected reinforcing scheme included a total of 643 reinforcing members. Reinforcing members were placed on a 3-ft by 3-ft (0.9-m by 0.9-m) staggered grid (with every other row offset by one half the spacing) over the area where previous slides had occurred. Additional reinforcing members were placed on a coarser 3-ft by 6-ft (0.9-m by 1.8-m) grid above the slide area to reduce the potential for future sliding in the upper portion of the slope. The factor of safety for the selected reinforcement configuration is estimated to be between 1.15 and 1.50.

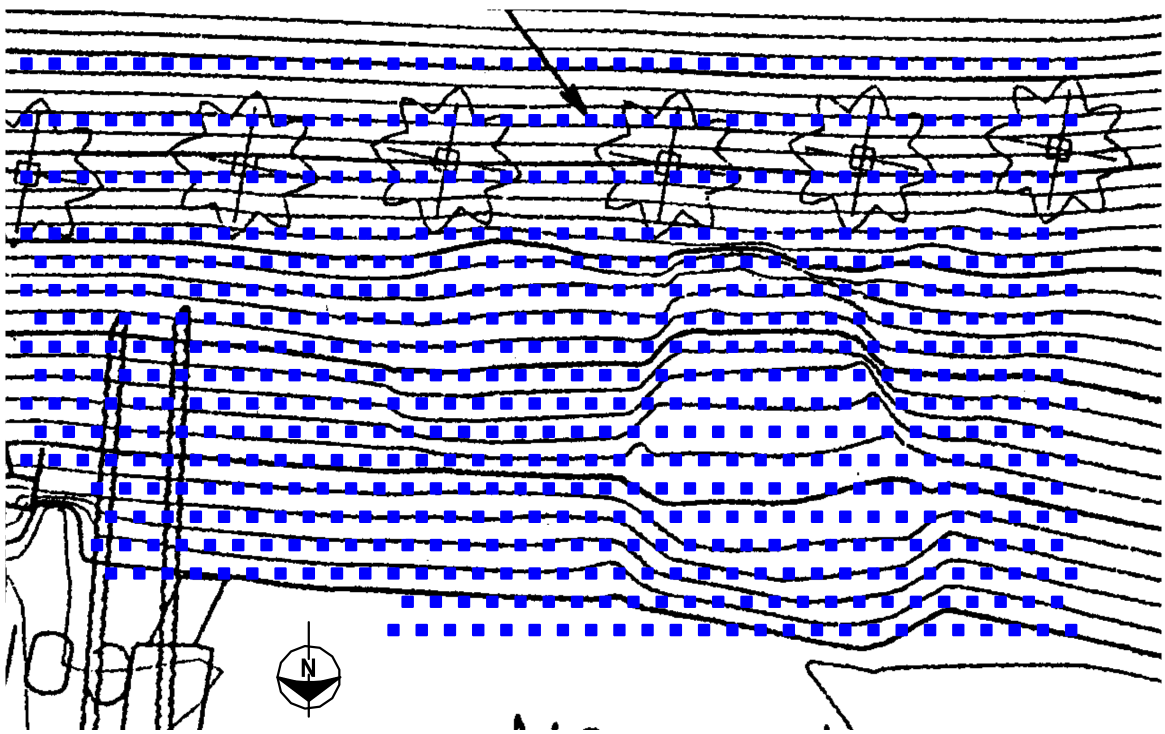
5.2.2. Stabilization Scheme for I435-Holmes Road Site

The I435-Holmes Road slope was stabilized using 3.5-in diameter galvanized steel pipe with 0.188-in (0.48-cm) thick walls. For design purposes, these members are considered to be “strong” members in the sense that the limit resistance for the members is controlled entirely by the strength of the soil rather than by both the strength of the soil and the strength of the reinforcing members as is generally the case for recycled plastic members.

These members were therefore selected to compare the overall effectiveness of the respective member types for installation and stabilization.



(a) cross-section view



(b) plan view

Figure 5.7 Selected stabilization scheme for the I435-Wornall Road site: (a) cross-section and (b) plan view superimposed on elevation contours of the site established following the most recent slide.

Figure 5.8 shows the assumed design cross-section for the I435-Holmes Road slope. Because no site specific soil property data was available prior to stabilization of the I435-Holmes Road site, no specific stability analyses were performed to select a stabilization scheme for this slope. Rather, the stabilization scheme was selected based on previous experience at the I70-Emma site during Phase I of the project and at the I435-Wornall Road site. The selected stabilization scheme, shown in Figure 5.9, generally consisted of reinforcement placed on a 3-ft by 3-ft (0.9-m by 0.9-m) staggered grid similar to that used at the I70-Emma site during Phase I and at the I435-Wornall Road site. However, given that the previous sliding surface was believed to be greater than 8-ft (2.4-m) deep near the center of the slope, three rows of reinforcement were left out of the grid to reduce costs while still maintaining a reasonable degree of stability against shallower slides in this portion of the slope. The selected stabilization scheme included a total of 276 reinforcing members. Most of these members were to be installed vertically while three of the top four rows of members were inclined perpendicular to the slope to ensure that they passed beneath the observed sliding surface.

Table 5.3 Summary of factors of safety determined for different reinforcement configurations and stability cases for the I435-Wornall Road site.

Reinforcement Spacing (ft)	Factor of Safety for Respective Stability Case				
	A	B	C	D	E
3L x 3T[1]	1.50	1.44	1.29	1.28	1.28
3L x 6T	--	--	1.14	1.20	1.12
3L x 3T in middle third; 3L x 6T elsewhere	--	--	1.17	1.31	1.19
3L x 3T in middle third only	--	--	1.09	1.08	1.13
3L x 3T in upper third only	--	--	--	1.00	1.00
3L x 3T in lower third only	--	--	--	1.00	1.00
3L x 6T in upper third only	--	--	--	1.06	1.06
3L x 6T in lower third only	--	--	--	1.05	1.10

[1] L and T denote spacing in longitudinal (strike) and transverse (dip) directions, respectively

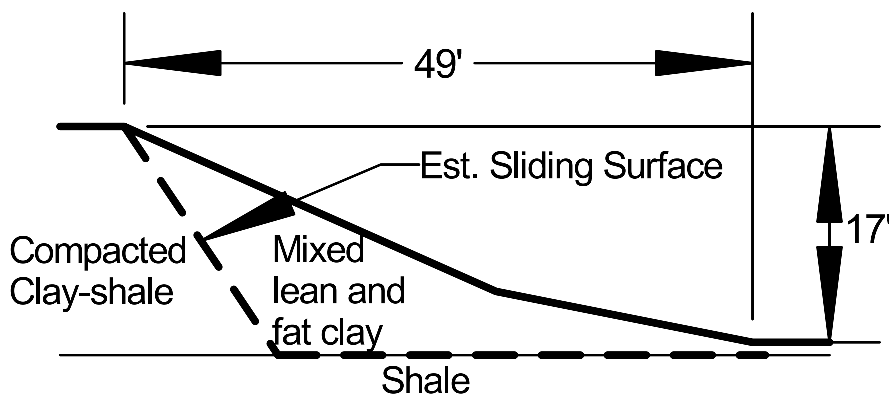
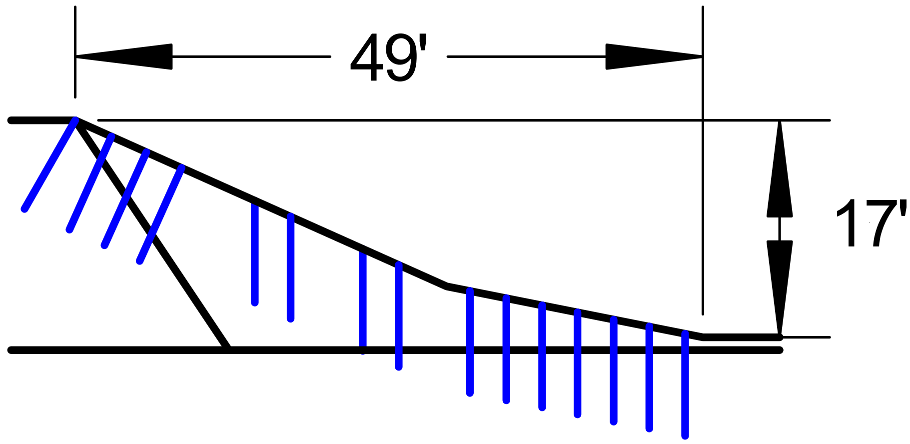


Figure 5.8 Assumed design cross-section for I435-Holmes Road slope.

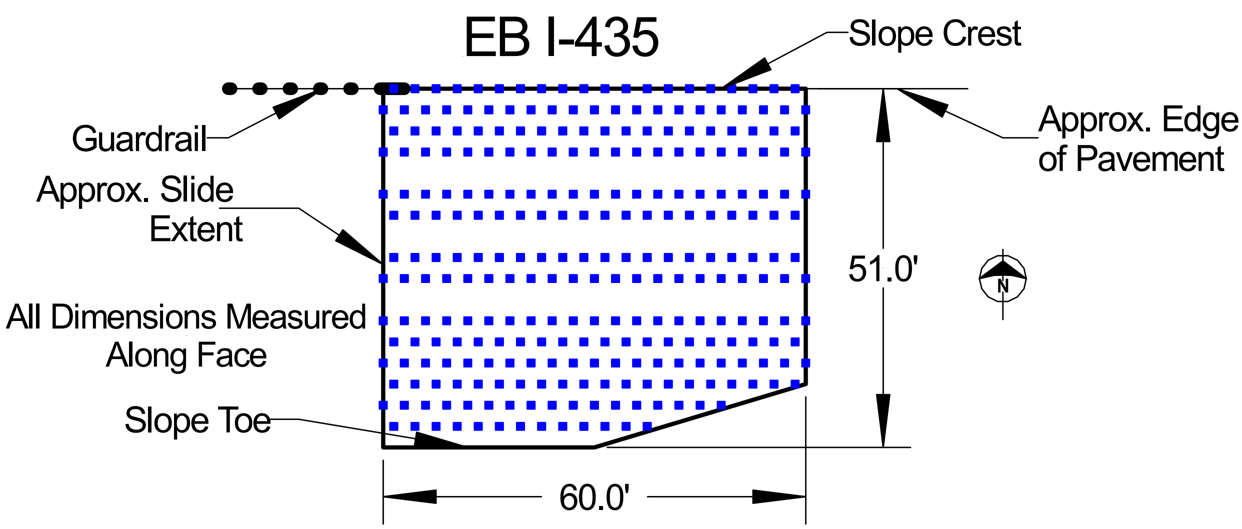
5.3. Field Installation

Field installation at the I435-Wornall Road test site began in October 2001 and was completed in December 2001. Installation at the I435-Holmes Road test site occurred during

December 2001. The following sections describe installation activities at each of the respective test sites.



(a) cross-section view



(b) plan view

Figure 5.9 Selected stabilization scheme for the I435-Holmes Road site: (a) cross-section and (b) plan view.

5.3.1. Installation at I435-Wornall Road Test Site

The I435-Wornall Road site was regraded to the original slope configuration by MoDOT maintenance crews in early October 2001. Installation of reinforcing members began on October 18, 2001 following several moderate to heavy rainfall events. At this time, noticeable seepage was observed coming from the slope in several locations and several small cracks were observed in the former slide area, which appeared to be an early indication of sliding. Furthermore, the available supply of reinforcing members at this time was not sufficient to complete the installation and additional members were not expected for several weeks. To respond to these observations, the installation plan was modified slightly so that alternating sections of reinforcing members would be installed to provide immediate stabilization across the slide area. These members were installed during the period October

18-30, 2001. The remaining sections of the stabilization pattern were then completed during the period December 5-7, 2001 after additional reinforcing members were acquired. Figure 5.10 shows a photograph of the slope at the completion of field installation. The survey flags in the photograph show the locations of all reinforcing members. Following the installation, the ornamental vegetation was replaced across the site and 4- to 6-in (10- to 15-cm) of landscaping mulch was placed across the slide area.

Figure 5.10 Photograph of I435-Wornall Road site following installation.

The equipment utilized for driving the reinforcing members at the I435-Wornall Road site included a Davey-Kent DK100B track-mounted hydraulic rig and a Ingersoll Rand (IR) CM150 pneumatic rock drill shown in Figure 5.11. A cable and pulley system was developed to assist maneuvering of both rigs on the slope and to prevent tipping of the rigs on the relatively steep slope (2.2H:1V). The Davey-Kent rig was previously utilized at the I70-Emma site during Phase I of the project with a great deal of success. However, significant problems were encountered in traversing the wet areas of the I435-Wornall Road slope with the Davey-Kent rig due to is relatively heavy weight, which caused severe rutting and resulted in the rig becoming stuck on several occasions. The lighter IR rig was therefore used to install the vast majority (590 out of 620) of reinforcing members installed at the site because of its lighter weight and additional maneuverability.

A total of 620 reinforcing members were installed in the slope[1]. Of these, 424 members were from Batch A4, which consisted of compression molded members with a relatively high filler content (primarily sawdust). The remaining members installed at the site were extruded members including 188 members from Batch A5, 3 members from Batch C9, 1 member from Batch B7, and 1 member from Batch B8. In addition to these members, three 3.5-in (9-cm) diameter steel pipe members (schedule 40) were also installed to evaluate the drivability of these members as compared to the recycled plastic members. Members in

[1] Several members originally included in the design layout were not installed because they fell below the toe of the slope or because they may have impacted a culvert running through the embankment on the eastern edge of the stabilized area.

the 3-ft by 3-ft (0.9-m by 0.9-m) staggered grid were generally driven vertically while members in the 3-ft by 6-ft (0.9-m by 1.8-m) grid at the top of the slope were generally installed perpendicular to the slope face. Four of the recycled plastic members installed at the site were instrumented members as described in more detail in Section 5.4.

(a) Davey-Kent DK100B track-mounted hydraulic rig

(b) Ingersoll Rand CM150 track-mounted pneumatic rig

Figure 5.11 Equipment used to install reinforcing members at the I435-Wornall Road site: (a) Davey Kent rig and (b) Ingersoll Rand rig.

A second problem encountered during installation was that penetration rates for the reinforcing members dropped dramatically when the reinforcing members encountered the stiff compacted clay shale fill. Depths to the stiffer clay shale varied from approximately 5-ft (1.5-m) at the toe of the slope to greater than 8-ft (2.4-m) at the crest of the slope. To avoid inflicting significant damage to the reinforcing members, installation was halted when the penetration rate dropped below approximately 3-in per minute (1.2-cm/min). At the same time, every effort was made to ensure that at least 6-in (15-cm) of penetration was achieved after reaching the stiffer soil to ensure that adequate resistance to sliding was available. The lengths of reinforcing members remaining above ground were removed using a gasoline-powered chain saw.

Figure 5.12 shows a frequency distribution of the average penetration rate, calculated as the total length of penetration divided by the total time of penetration, for 502 of the reinforcing members installed at the I435-Wornall Road site. All members included in the distribution are from Batches A4 and A5. As shown in the figure, penetration rates ranged from 0.5- to 14-ft/min (0.2- to 4.2-m/min) with an average penetration rate of 5.4-ft/min (1.6-m/min). This corresponds to an average driving time of 1.5 minutes for 8-ft (2.4-m) long members. Penetration rates for limited numbers of recycled plastic members from other batches (B7, B8, and C9), as well as steel pipe members, produced penetration rates that were similar to those observed for members from Batches A4 and A5 as shown by the arrows in Figure 5.12. The similarity in penetration rates for members with widely different stiffness suggests that member stiffness has a limited influence on penetration rate. Additional details regarding the relative penetration rates observed for different types of members can be found in Bowders et al (2003).

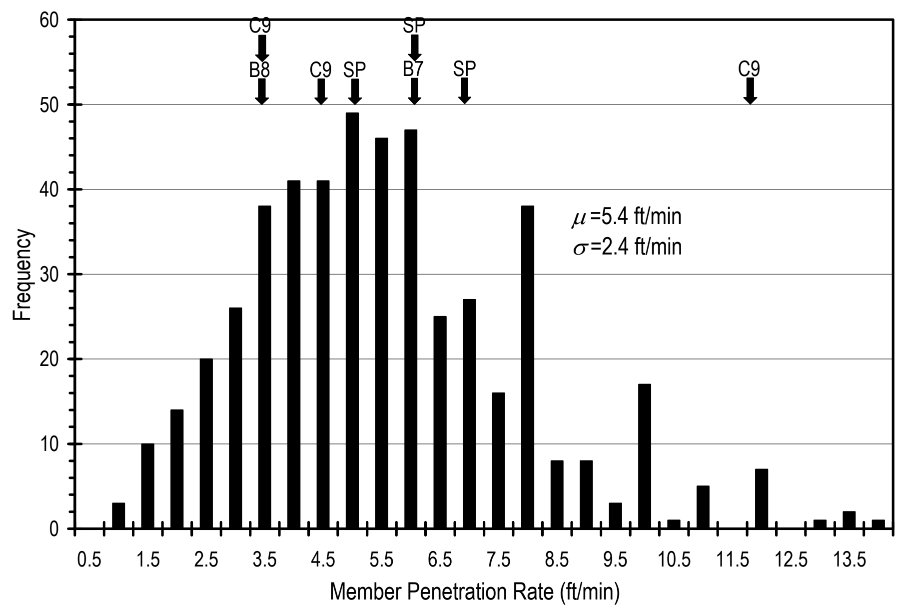


Figure 5.12 Frequency distribution of average penetration rates for recycled plastic members from Batches A4 and A5 installed at the I435-Wornall Road site. (μ=mean, σ=std. dev.)

Installation rates – the rate of installation including "set up" time between members – varied dramatically during installation of the reinforcing members. Several modifications to the driving equipment were evaluated during the first few days of installation. Installation rates during these trials were generally very low because of the trial and error process required for evaluating the equipment. After several days of trials, installation rates increased dramatically and approached the installation rates that were achieved at the I70-Emma site during Phase I. The peak installation rate achieved at the I435-Wornall Road site was 114 pins per day with an average installation rate of approximately 80 pins per day (excluding the initial trials undertaken during the first days of installation).

5.3.2. Installation at I435-Holmes Road Test Site

Installation at the I435-Holmes Road site was performed during the period December 14-20, 2001 using the same IR CM150 rig used for installation at the I435-Wornall Road site. A total of 262 members[2] were installed at the site including 256 galvanized steel pipes and 6 recycled plastic members from Batch A5 for comparative purposes. Two of the steel pipe members were instrumented as described in Section 5.4.

Figure 5.13 shows a photograph of the site just after completion of the installation. Unlike the I435-Wornall Road site, the I435-Holmes Road site was not regraded to its original slope prior to installation. Instead, reinforcing members were installed such that the top of the members would lie at the anticipated ground surface after the site was regraded. Members installed near the crest of the slope generally did not meet refusal while members installed in the middle and lower portions of the slope met refusal at depths ranging from 4- to 8-ft (1.2- to 2.4-m). This suggests that the compacted clay shale stratum was shallower than originally assumed for selection of the stabilization scheme. Members that could not be installed to the requisite depth were cut off at the appropriate location using an acetylene torch or a portable band saw. Following installation, any void space within the pipes that did not become plugged with soil during installation was filled with bagged cement grout to prevent accumulation of water within the pipes. The upper portion of the slope, near the existing slide scarp, was then regraded by MoDOT maintenance crews at which point the ornamental vegetation on the slope was replaced.

Penetration rates were recorded for 218 of the steel pipes and all 6 plastic members. As shown in Figure 5.14, the average penetration rate for the steel pipes was 5.0-ft/min (1.5-m/min) with a standard deviation of 2.1-ft/min (0.6-m/min) while the average penetration rate for the plastic members from Batch A5 was 4.6-ft/min (1.4-m/min), only slightly lower than that observed for the steel members. These observations again suggest that member stiffness has only a limited effect in determining field installation rates. The peak installation rate (including set up time between members) was again near 100 members per day, with an average installation rate of approximately 80 members per day.

Overall, use of steel reinforcing members provided little benefit over use of recycled plastic members in terms of constructability. Penetration rates observed for the steel members were slightly greater than those observed for recycled members. However, the steel members are significantly heavier (approximately 70-lb.) than recycled plastic members (approximately 45-lb.) and are more difficult to cut off when refusal is met during

[2] Several members included in the original design layout were again eliminated during field layout and installation because they fell beyond the extent of the slide.

installation. As a result, overall installation rates observed for steel and recycled plastic members were comparable. Thus, there appears to be little advantage to using steel members from an installation point of view, although there may be some advantages (or disadvantages) in terms of their effectiveness for long-term stabilization.

Figure 5.13 Photograph of I435-Holmes Road site just after installation.

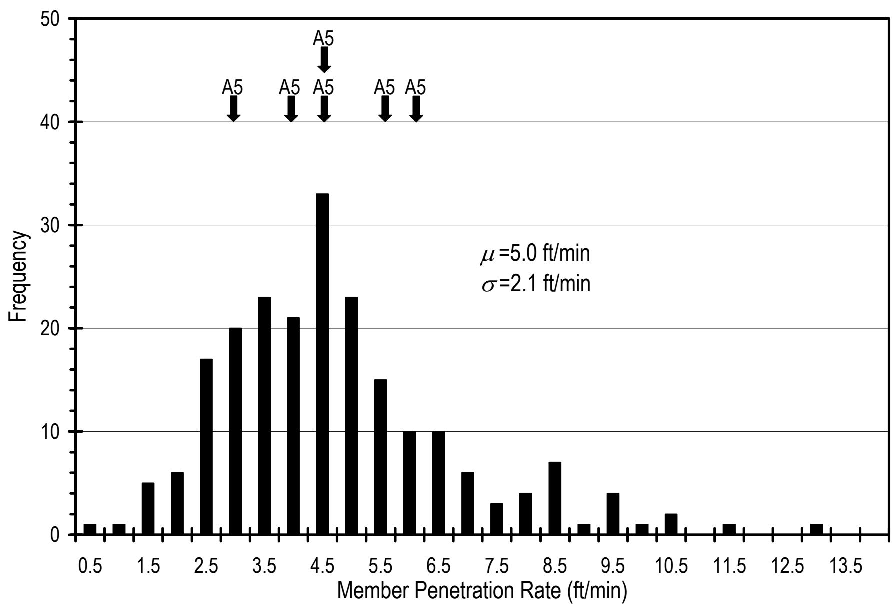


Figure 5.14 Frequency distribution of average penetration rates for galvanized steel members installed at the I435-Holmes Road site. (μ=mean, σ=std. dev.)

5.4. Instrumentation

A variety of different types of instrumentation was installed in each of the slopes to monitor the performance of the stabilized and control slopes as well as to gather data to establish the loads mobilized in selected reinforcing members. In this section, the different types of instrumentation utilized at the I435-Kansas City and other test sites are described along with methods used for calibration, analysis, and interpretation of the different types of instrumentation. The specific instrumentation schemes used for each slope within the I435-Kansas City test site complex are then presented.

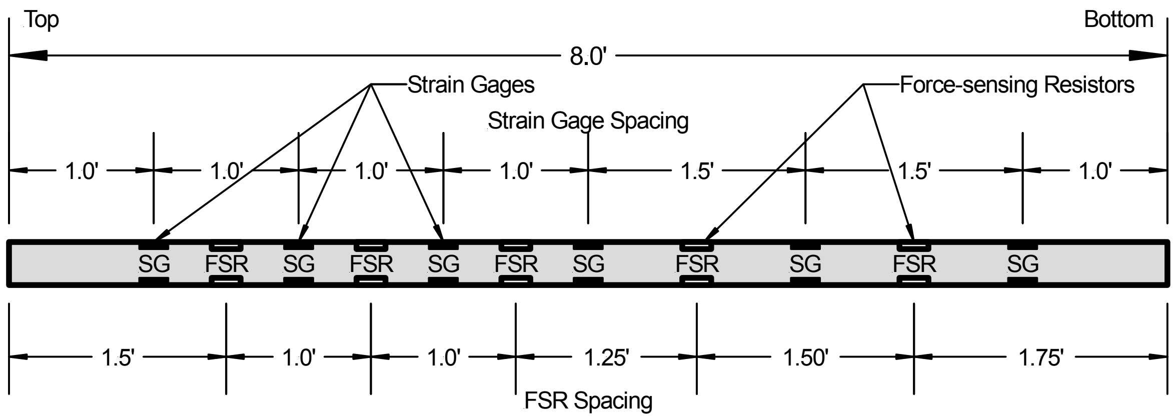
5.4.1. Types of Instrumentation

Several different types of instrumentation were installed in the three slopes within the I435-Kansas City test site complex. Specifically, instrumentation was installed to monitor lateral deformation, pore water pressure, and moisture content within the slopes at selected locations; selected reinforcing members were also instrumented to monitor the loads developed in the members. Lateral displacements were monitored with conventional slope inclinometers. Pore water pressures were monitored with conventional standpipe piezometers screened at selected depths. In addition, moisture content or soil suction was monitored using three different types of devices: ThetaProbes®, Equitensiometers®, and Profile Probes®. Loads in the reinforcing members were monitored using electrical resistance strain gages and "force-sensing resistors" (FSR). Details of each type of instrument are provided in the following sections.

Measurement of lateral deformations. Lateral deformations were measured at each site using conventional slope inclinometers. Standard 2.5-in (6.4-cm) diameter inclinometer casing was installed in 6-in (15-cm) diameter boreholes and backfilled with clean pea gravel. Larger 6-inch (15-cm) diameter boreholes were used to avoid potential problems with backfilling based on experience from the I70-Emma site during Phase I. Where possible, each casing was extended to approximately 5-ft (1.5-m) below the toe of the slope. In cases where very stiff soil was encountered at shallower depths, the casing was extended at least 3-ft (0.9-m) into the stiff soil to ensure adequate founding of the inclinometer casings. Following installation, lateral deformations were regularly measured using an inclinometer probe provided by MoDOT.

Measurement of loads in instrumented reinforcing members. Several members within the stabilized slopes were instrumented with strain gages and force-sensing resistors (FSR) to monitor the loads mobilized in the reinforcing members. Figure 5.15 shows a schematic of the instrumented recycled plastic reinforcing members installed in the stabilized slopes and a photograph of a 4-ft (1.2-m) long "test" member containing these sensors. Each instrumented member contained six pairs of strain gages placed on opposite (uphill and downhill) sides of the member. Strain gage pairs were placed at 1-ft (0.3-m) intervals over the top 4-ft (1.2-m) of each member and at 1.5-ft (0.45-m) intervals below this point. Five pairs of FSR were also placed on opposite sides of the members halfway between each pair of strain gages. Several steel pipe members were also instrumented with a similar array of strain gages but without the FSR.

The strain gages used are the electrical resistance type similar to those frequently used to measure strains in concrete and steel members for structural applications. These gages have the advantage of being inexpensive and commercially available with strain ranges suitable for the project. The disadvantage of this type of gage is that they are not generally well-suited for long-term monitoring, particularly in a buried environment. Several alternative types of strain gages were also considered including vibrating-wire and fiber-optic strain gages. However, these gages are substantially more expensive than electrical resistance gages and are not available with strain ranges needed for the project. There was also significant concern regarding whether these gages could survive installation. The decision was therefore made to use relatively large numbers of electrical resistance type gages with the hope that a sufficient number of them would survive long enough to enable conclusions to be drawn regarding the loads in the members.



(a) schematic

(b) photograph

Figure 5.15 Instrumentation used to monitor loads in recycled plastic reinforcing members: (a) schematic of instrumented member and (b) photograph of an instrumented test member.

The strain gages used to instrument the recycled plastic members were generally 350-Ohm electrical resistance gages (Vishay Measurements Group part number EP-08-500BL-

350) with a working range of 15000 microstrains and a gage length of 0.5-in (1.3-cm) as shown in Figure 5.16. Several 120-Ohm gages (part number EP-08-500AF-120) with similar characteristics were also used for selected members. Gages were placed on the recycled plastic members in 0.125-in (0.3-cm) deep recesses machined into the members to prevent the gages from being ripped off during installation. Gages were attached using a special adhesive (M-bond Type AE resin with Type 15 curing agent) selected to be compatible with the plastic members. The adhesive requires curing at elevated temperatures while the gages are under a nominal normal load. The gages were therefore applied to one side of the member at a time and then placed in a controlled temperature box for curing as shown in Figure 5.17. Following curing, all gages were wired and the wires placed in machined grooves running along the neutral axis of the member. The gages were then sealed with Vishay Measurements Group 3145 RTV sealant (for insulation and waterproofing) and the recessed areas for the gages and wires filled with common silicon caulk to provide a secondary seal against moisture and to hold the wires in place.

Figure 5.16 Photographs of 350-Ohm electrical resistance strain gages used to monitor strains in the recycled plastic reinforcing members.

The strain gages used for the steel members were 350-Ohm electrical resistance gages (part number CEA-06-250UN-350) with a strain range of 5000 microstrains and a gage length of 0.25-in (0.64-cm). The gages were attached using conventional adhesives that cure at room temperature. Since recessed areas for the strain gages could not be efficiently made, the gages were attached to the surface of the steel pipe and covered with 0.5-in (1.3-cm) angle sections as shown in Figure 5.18, which were tack welded to the pipe to protect the gages during installation.

In addition to the strain gages, each of the instrumented recycled plastic members was also fitted with several “force-sensing resistors”, or FSR. These sensors are thin, 1.5-in by 1.5-in (3.8-cm by 3.8-cm) square electrical pads shown in Figure 5.19 that have a resistance that is proportional to the force applied to the sensor. These sensors are commonly used in touch pads for automated teller machines and other similar equipment. They are not intended for use as pressure sensors. However, they do have a resistance that is proportional to the applied pressure and thus can provide at least a qualitative measure of the lateral pressures being imposed on the reinforcing members. Experience gained from analyzing the strain gage data from the I70-Emma site during Phase I (where FSR were not used) demonstrated the benefit that could be gained by having some measure of the distribution of lateral pressures, even if only a qualitative measure. Several different types of total stress cells were investigated for this purpose. However, all available cells were generally too large for use on the recycled plastic members and were generally believed to be cost prohibitive. Given that the FSR are inexpensive (approximately $5/each) it was decided to try these sensors in the hope that they could provide some information during analysis and interpretation of the strain gage data.

Figure 5.17 Photographs of temperature controlled box utilized for curing the adhesive used to attach strain gages to recycled plastic members.

Figure 5.18 Photograph of instrumented steel pipe member with 0.5-in angle section to protect gages during installation.

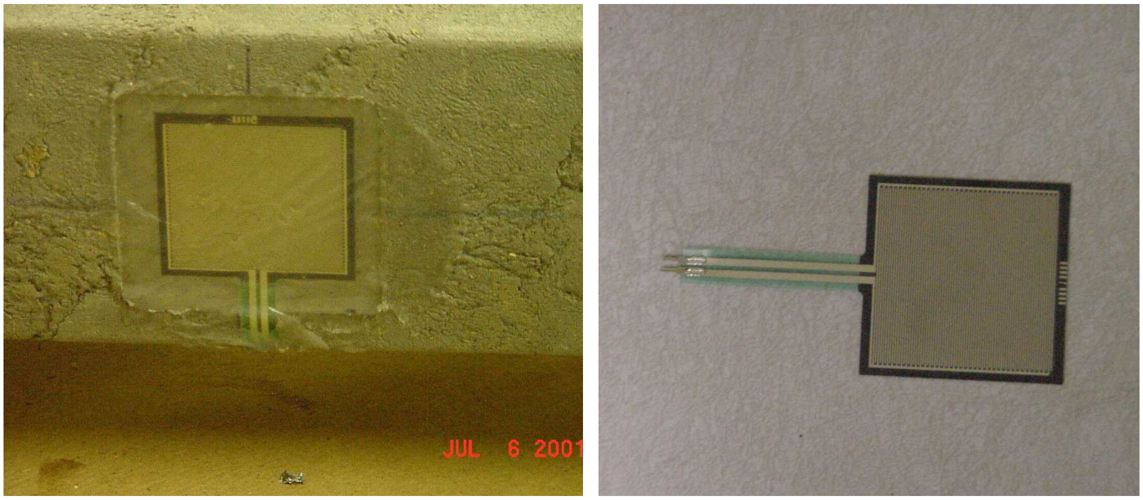


Figure 5.19 Photograph of force-sensing resistor used to measure applied pressure on instrumented recycled plastic members.

Since the sensors are intended to measure normal loads, it is not necessary to firmly bond the FSR to the member. FSR were therefore simply applied to the members using the adhesive backing that comes on the sensors from the manufacturer. The sensors were placed in 0.0625-in (0.16-cm) deep recesses, wired for connection to the data acquisition system, and then covered with a thin layer of common silicon caulk to provide a seal against water. Wiring for the sensors was placed in machined grooves and caulked into place in a manner similar to that for the strain gages.

Readings for the strain gages and FSR were acquired using the data acquisition system shown in Figure 5.20 that was specifically designed and built for this project. This system permits readings for all strain gages and FSR for a member to be taken in approximately one minute as compared to the 60 minutes required to read all sensors for a member manually. The system has therefore permitted more frequent readings to be taken and permitted readings from several sites to be taken in a single day.

The data acquisition system measures a voltage differential across each sensor. In the case of the strain gages, the voltages are directly proportional to the strain in the sensors and thus can be directly converted to strain. In the case of the FSR however, it is necessary to determine resistance in each sensor prior to converting that resistance to an applied pressure. To do this, the data acquisition system was outfitted with a series of six fixed resistors with resistances of 100, 1000, 5000, 10000, 50000, and 100000 ohms, respectively. The voltage differential across each of these fixed resistors was then measured each time a set of readings was taken to establish the relation between measured voltage and known resistances for that set of readings. Measured voltages for each FSR were then converted to resistances using a non-linear least-squares fit of the voltage-resistance relation for that particular reading. Figure 5.21 shows the relation between voltage and resistance determined in this manner for one set of readings.

Once the resistance for each sensor is determined, the corresponding pressure is established based on the relation between resistance and applied pressure determined in several laboratory calibration tests. The calibration was established by loading several FSR in a laboratory loading apparatus over pressures ranging from 8- to 96-psi (55- to 660-kPa). The laboratory loading apparatus, shown in Figure 5.22, consisted of an FSR mounted on a piece of recycled plastic and covered with a thin layer of common silicon caulk to mimic the conditions used to mount the sensors on the reinforcing members. The FSR and plastic piece were then placed within a 4-in (10-cm) diameter PVC confining ring and covered with approximately 0.75-in (1.9-cm) of concrete sand. The entire assembly was then placed in a load frame and loaded incrementally. Resistance readings were determined for each load for several sets of tests including several load-unload cycles.

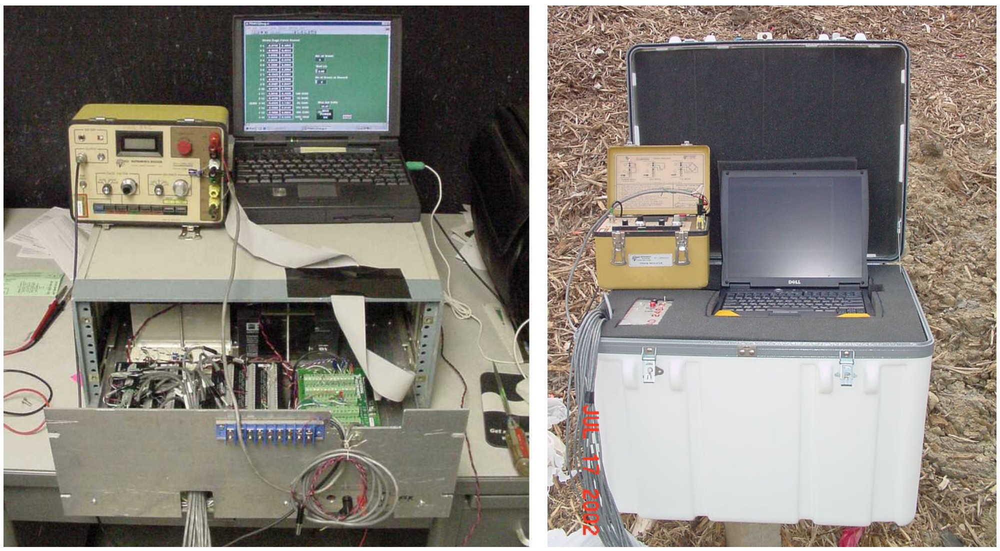


Figure 5.20 Data acquisition system developed, constructed, and used in this project to acquire readings from instrumented reinforcing members.

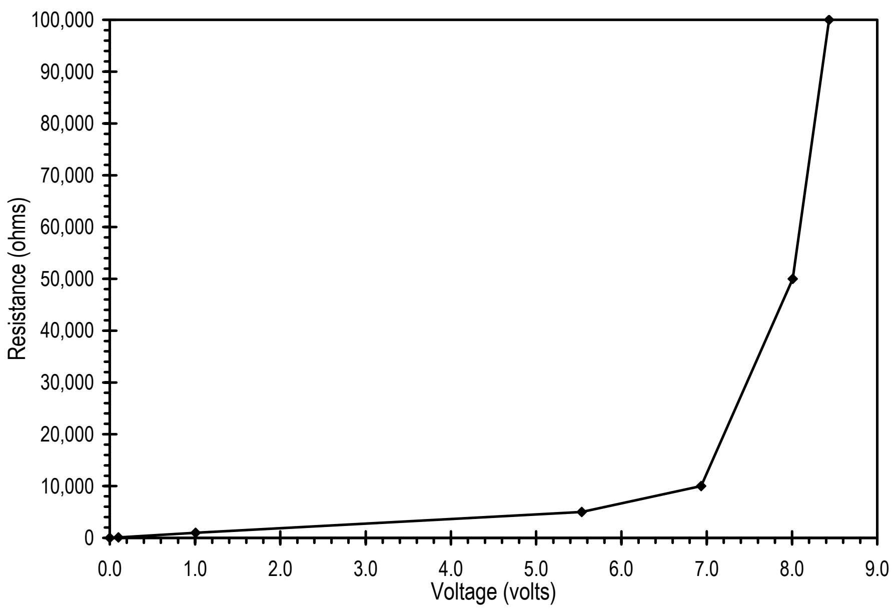


Figure 5.21 Sample of relation between measured voltage and resistance for fixed resistors in data acquisition system.

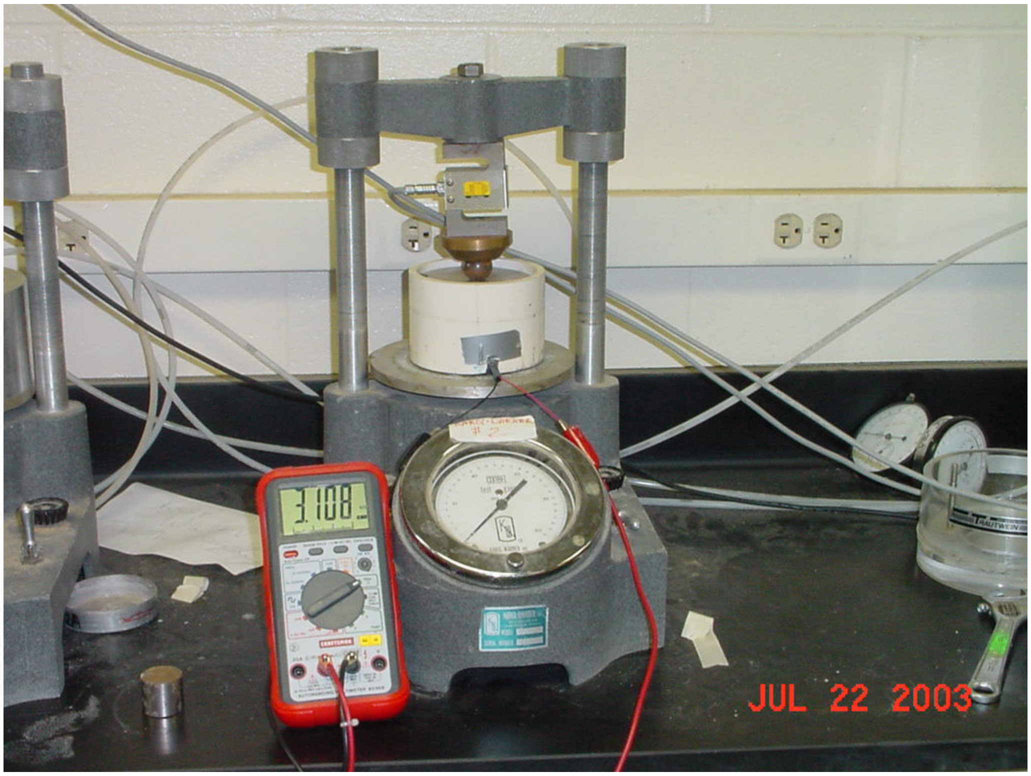


Figure 5.22 Apparatus used to develop calibration for FSR.

Figure 5.23 shows the results of the calibration tests along with the least-squares relation used to convert measured resistance to applied pressure. At relatively high pressures, very little scatter was observed in the data and the relation between applied pressure and resistance is very well defined. The amount of scatter increases with decreasing applied pressure, however, and at pressures less than 1000-psf (48-kPa) the scatter is so great that it become very difficult to establish the resistance-pressure relation. Based on these results, a "detection limit" for the sensors was set at 100-ohms. Field resistance readings above this value are therefore considered to be above the detection limit, in which case a value for applied pressure can not be determined.

Measurement of pore pressures, moisture content, and soil suction. Water conditions within each of the slopes were monitored using several types of instrumentation in an attempt to handle the range of possible conditions that might exist in the slopes. Common standpipe piezometers were installed to monitor positive pore water pressures within each slope. Negative pore water pressure, or soil suction, was monitored using several different types of sensors to directly measure soil suction, or to measure soil moisture content, which can be related to soil suction through a soil-water characteristic curve (SWCC).

The standpipe piezometers used at the field sites were generally placed in clusters of two or three standpipes within a single borehole. Each standpipe was screened over different 1- to 2-ft (0.3- to 0.6-m) depth intervals with the intervals hydraulically isolated using bentonite plugs as shown in Figure 5.24. The standpipes were constructed at the MU Geotechnical Engineering laboratories using 0.75-in diameter PVC pipe that was slotted at the requisite locations and then covered with a non-woven geotextile. Standpipe piezometers were then installed at the sites by MoDOT drilling crews with assistance from MU researchers.

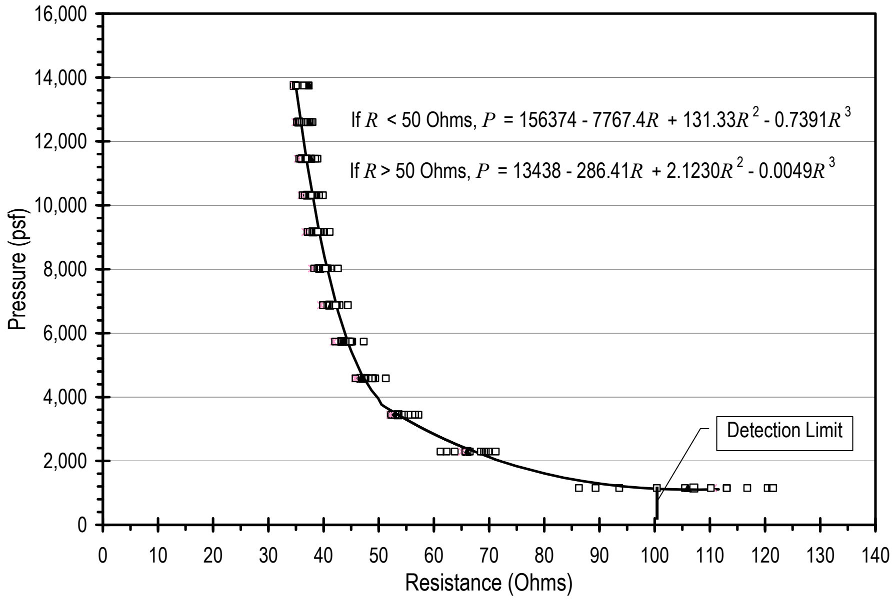


Figure 5.23 Calibration curve for FSR relating measured resistance to applied pressure.

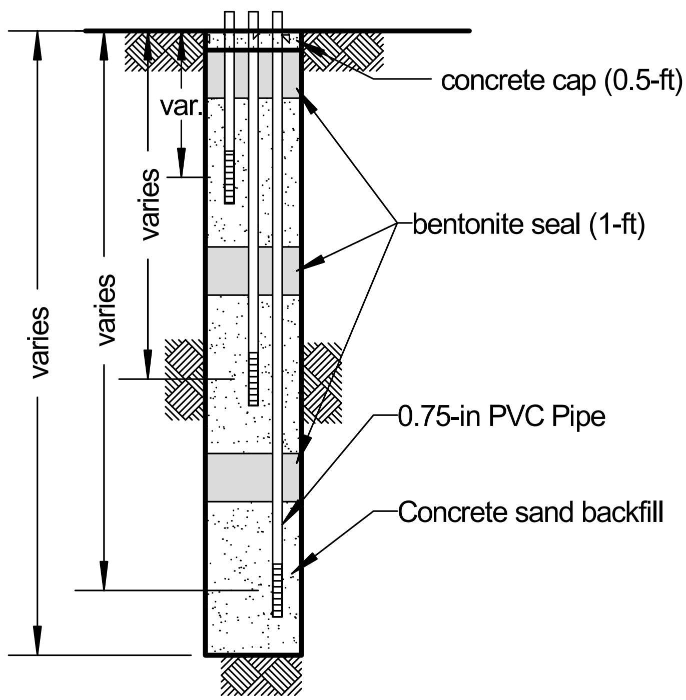


Figure 5.24 General schematic for standpipe piezometers installed at field test sites.

In addition to standpipe piezometers, two types of sensors, known as ThetaProbes® and Equitensiometers®, were installed to continuously monitor the water conditions within the slope. The ThetaProbe®, shown in Figure 5.25, consists of four sharpened stainless steel rods that are connected to a waterproof housing which contains the sensor electronics. The probe measures volumetric soil moisture content by responding to changes in the soil's apparent dielectric constant and converting these changes into a DC voltage that can be correlated to volumetric moisture content. The probe responds to the changes in the dielectric constant by generating a 100 MHz sinusoidal signal that extends into the soil by means of the array of four stainless steel rods. A voltage standing wave is formed along an internal transmission line of the probe from the reflection of the 100 MHz signal, which is affected by the impedance of the rod array. The impedance of the rod array fluctuates with the impedance of the soil, which is a function of the soil's dielectric constant and ionic conductivity. The 100 MHz signal minimizes the effects of ionic conductivity, so that the variation in the impedance is almost solely due to the soil's apparent dielectric constant. Dry soils typically have a dielectric constant between 3 and 5. The dielectric constant of water is approximately 81 and air is 1. Since the dielectric constant of dry soil is typically much less than water, the dielectric constant of soil is primarily determined by its water content. Published studies have shown that there is nearly a linear correlation between the square root of the dielectric constant and volumetric moisture content for most soil types. The volumetric moisture content is the ratio between the volume of water present and the total volume of the sample, expressed either as a percentage of volume (%vol) or a ratio ($m^3.m^{-3}$). Pure water has a ratio of 1.0-$m^3.m^{-3}$ and a completely dry soil would have a ratio of 0.0-$m^3.m^{-3}$. The output signal of the ThetaProbe® is 0- to 1-V DC for a range of soil dielectric constant between 1 and 32, which corresponds to a volumetric soil moisture content of approximately 0.5-$m^3.m^{-3}$. Installation of the ThetaProbe® is simple. The probe is inserted in the soil until the rods are completely covered. The probes are designed to be installed either at the ground surface or buried in a borehole or trench that is carefully backfilled.

The Equitensiometer® probe, shown in Figure 5.25, is essentially a ThetaProbe® in which the four measuring rods are embedded in a porous material that serves as an equilibrium body. Equitensiometers® measure soil matric potential, which can be thought of as the negative pressure or soil suction required to extract water from between the soil particles. The porous material surrounding the four rods has a known stable relationship between water content and matric potential (i.e. a known SWCC). When the probe is inserted into the soil, the matric potential of the porous material quickly equilibrates to that of the surrounding soil and the water content of the equilibrium body is measured by the rods of the ThetaProbe®. The measurement recorded by the ThetaProbe® can then be converted into the matric potential of the surrounding soil using calibration curves supplied by the manufacturer for each probe. Installation of the Equitensiometers® requires more care than the ThetaProbe®. The porous material of the Equitensiometer® needs to be thoroughly saturated before the probe is installed and the probe must be installed in a horizontal or slanting angle. Vertical installation of the probe could lead to non-representative readings due to water running down the side of the probe and excessively wetting the surrounding soil. Small changes to the soil structure surrounding the probes should not affect the accuracy of the readings, however, the probes must be in firm contact with the surrounding soil or any gaps filled with a quartz powder suspension to allow the porous material of the probe to properly equilibrate with the surrounding soil. The probes are designed to withstand prolonged installation periods with little maintenance if installed correctly.

Figure 5.25 Instrumentation used to monitor soil water conditions at the I435-Kansas City site and other sites: ThetaProbe® ML2x (bottom left), Equitensiometer® EQ2 (bottom right), THLog® data logger (top center), Profile Probe® PR1/6 and access tube (center), and HH2 readout unit (lower center).

At each test site, two ThetaProbes® and two Equitensiometers® were installed in a vertical cluster to monitor the soil moisture conditions near the ground surface. The probes were installed in a 3.3-ft (1-m) deep trench as illustrated in Figure 5.26. The probes were installed in the vertical up-slope face of the trench in alternating succession at depths of approximately 8-in (20-cm), 16-in (40-cm), 24-in (60-cm), and 40-in (100-cm). To install each sensor, a 2.5-in (6.4-cm) sampling tube was inserted horizontally approximately 6 inches (15-cm) into the vertical face of the trench to create a hole for the probes. ThetaProbes® were pushed into the end of the hole until the rods were fully inserted into the soil, after which the space surrounding the probe was filled with onsite soil. The Equitensiometers® were first submerged in water for a minimum of 24 hours prior to installation in the slope. Equitensiometers® were then placed in the center of the installation hole and the hole was filled with a quartz powder suspension to ensure intimate contact of the probe and the surrounding soil. After all four sensors were installed, the sensors were connected to a THLog® data logger, which supplies power to the sensors and records measurements from the sensors at two hour intervals.

The final type of moisture instrumentation used is the Profile Probe® shown in Figure 5.25. The Profile Probe® works following the same philosophy as the ThetaProbe®, except that the source and receiver are rings on a slender probe, rather than spikes that are inserted into the soil. The Profile Probe® PR1/6 contains six sensor sets (source and receiver) to measure moisture content at depths of 4-, 8-, 12-, 16-, 24-, and 40-in (10-, 20-, 30-, 40-, 60-, and 100-cm) simultaneously. Measurements are taken by inserting the Profile Probe® into special composite "access tubes" that are installed in small, pre-drilled holes at the site. The pre-drilled holes are slightly smaller than the access tubes themselves to ensure intimate contact between the tubes and the surrounding soil. Once the access tubes are installed, measurements are taken by simply inserting a Profile Probe® into the access tube and using the HH2 readout unit to store readings for all sensor sets simultaneously as shown in Figure 5.28.

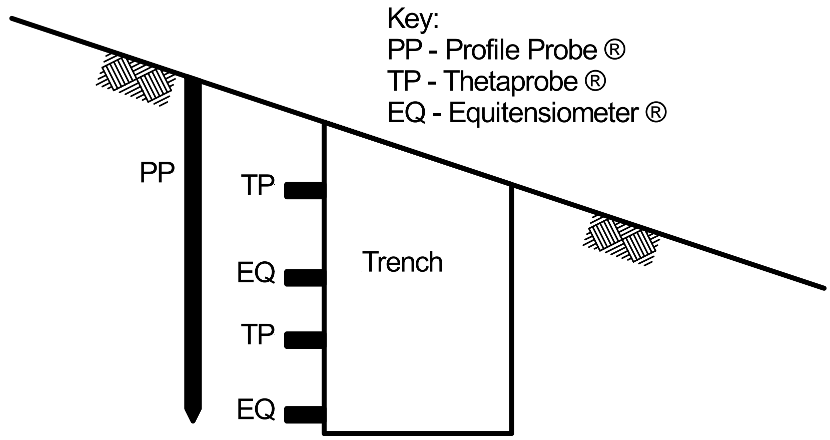


Figure 5.26 Schematic of ThetaProbe® and Equitensiometer® installation.

Figure 5.27 Photograph of installation of ThetaProbes® and Equitensiometers® in trench.

Figure 5.28 Photograph of Profile Probe® while taking readings.

5.4.2. Instrumentation for the I435-Wornall Road Site

A schematic showing the location of instruments installed at the I435-Wornall Road site is shown in Figure 5.29. The instrumentation at this site included four inclinometers, four instrumented reinforcing members, two clusters of standpipe piezometers, and an array of moisture instrumentation.

Instrumented reinforcing members were installed during installation of all reinforcing members. Two members, designated members IM-1 and IM-2, were installed near the center of the slide in close proximity to one another. Instrumented member IM-3 was installed downslope of IM-1 and IM-2 while member IM-4 was installed near the center of the western half of the slide area where sliding initiated during the previous failure. Each of the instrumented members was driven to full depth. Wiring from the instrumented reinforcing members was then buried in shallow trenches to extend to a weather resistant box located near the center of the site to provide protection for the electrical connections and a convenient location for connecting the data acquisition system.

Slope inclinometer casings were installed on November 27, 2001. Inclinometer I-1 was located near the center of the eastern half of the slide area at about the mid-point of the slope. Inclinometers I-2 and I-3 were placed near the center of the slide area in close proximity to instrumented reinforcing members IM-1, IM-2, and IM-3. Inclinometer I-4 was placed adjacent to instrumented member IM-4 near the center of the western half of the slide area. Each inclinometer casing was founded in the stiff clay shale but was not extended into the limestone layer lying beneath the shale. Approximate depths for the respective inclinometers are 19.0-ft (5.8-m) for I-1, 26-ft (7.9-m) for I-2, 14.5-ft (4.4-m) for I-3, and 19.5-ft (5.9-m) for I-4. All casings were cut off approximately 0.5-ft (15-cm) above the ground surface.

Two clusters of standpipe piezometers were also installed in the western half of the slide area on July 10, 2002. One cluster containing piezometers P-1 and P-2 was installed near the location of the head scarp of the previous slide. Piezometer P-1 is screened at a depth of 11.6-ft (3.5-m) and P-2 at 5.0-ft (1.5-m). The second cluster of piezometers is located downslope of the upper cluster near the lower third-point of the slope. In this cluster, piezometer P-3 is screened at 11.0-ft (3.4-m) and P-4 at 4.0-ft (1.2-m).

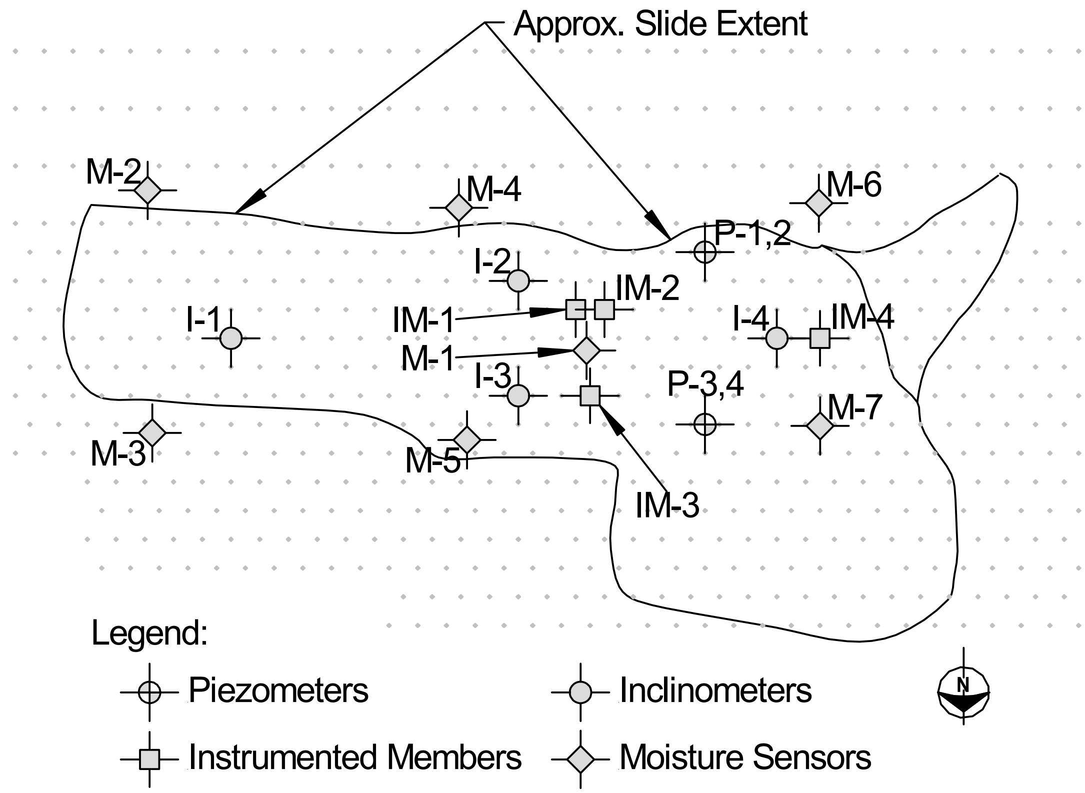


Figure 5.29 Instrumentation layout for I435-Wornall Road site.

Moisture instrumentation was installed at seven different locations, designated M-1 through M-7, on August 8, 2002. Profile Probe® access tubes were installed at each of these locations to measure the variation of moisture content with depth at each location at discrete time intervals as well as to establish the variability of moisture contents over the entire slide area. At location M-1, an array of two ThetaProbes® and two Equitensiometers® was installed in close proximity to the Profile Probe® access tube (Figure 5.26). ThetaProbes® were installed at depths of 8- and 24-in (20- and 60-cm) while Equitensiometers® were installed at depths of 16- and 40-in (40- and 100-cm). Each of these sensors were connected to a field data logger that permitted moisture content and soil suction readings to be taken at two hour intervals, thus enabling for essentially continuous measurement of moisture content and soil suction at location M-1.

5.4.3. Instrumentation for the I435-Holmes Road Site

A schematic of the instrumentation layout for the I435-Holmes Road site is shown in Figure 5.30. The instrumentation consisted of two instrumented steel pipe members, one inclinometer, one cluster of two piezometers, and two Profile Probe® access tubes.

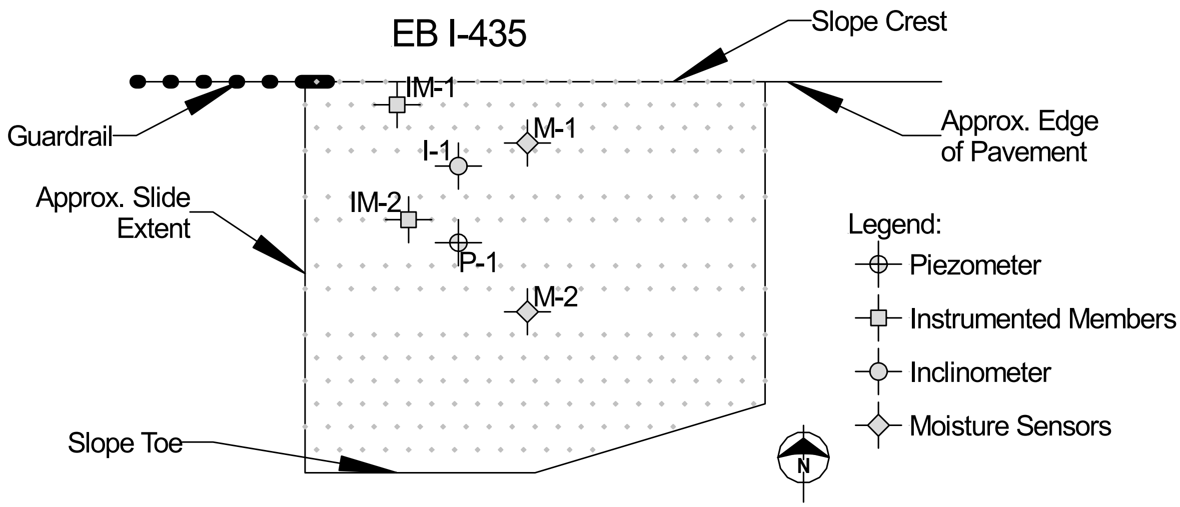


Figure 5.30 Instrumentation layout for I435-Holmes Road site.

The two instrumented members are located in the upper portion of the slope within the western half of the former slide area. Member IM-1 is an inclined member aligned roughly perpendicular to the slope face, while member IM-2 is inclined vertically. Because many of the members at this site could not be driven to full depth, the tips (bottom end) of the two instrumented members were cut off prior to installation to prevent having the members extend significantly above ground level. The length of the members that were cut off was determined from the measured penetrations of the surrounding members. Member IM-1 was cut to a length of 5.9-ft (1.8-m) while member IM-2 was cut to a length of 6.7-ft (2.0-m), eliminating the lowest pair of strain gages for both members.

The inclinometer and piezometers were installed by MoDOT drilling crews on July 11, 2002. The inclinometer casing was extended to a depth of approximately 18-ft (5.5-m) and founded in the stiff shale. The piezometers were installed within a single borehole and screened at different depths. Piezometer P-1 was screened at a depth of 2.5-ft (0.8-m); piezometer P-2 was screened at a depth of 13.0-ft (3.9-m). Profile Probe® access tubes were installed at two locations, designated M-1 and M-2, on October 9, 2002 to monitor water contents in the soils.

5.4.4. Instrumentation for the I435 Control Slope

Instrumentation for the I435 Control slope included one slope inclinometer and three Profile Probe® access tubes installed along a line passing through the center of the former slide area. The inclinometer casing was installed near the center of the slide area to a depth of approximately 20-ft (6.1-m) on July 11, 2002. The three access tubes were installed on October 9, 2002, with access tube M-1 installed near the toe of the slope, access tube M-2 near the midpoint of the slope, and access tube M-3 near the crest of the slope.

5.5. Field Performance

As of the writing of this report, the I435-Kansas City test sites have been in place for approximately 20 months. During this time, readings have been taken on all field instrumentation at intervals ranging from 1 to 4 months. These measurements have been processed, analyzed, and interpreted to produce the results presented in this section to demonstrate the performance of the stabilized slopes.

5.5.1. Precipitation at I435-Kansas City Sites

Figure 5.31 shows daily and monthly precipitation totals recorded at the weather station at Lee's Summit Municipal Airport, located approximately 4-miles (6.4-km) east of the test sites. Precipitation patterns observed since installation have been rather typical with relatively heavy precipitation during the Spring and significantly less precipitation throughout the rest of the year. Precipitation during the first few months after installation was generally limited with the exception of a single heavy precipitation event in late January 2002. Precipitation then increased substantially between April and June 2002 with numerous heavy precipitation events and large monthly precipitation. Precipitation then decreased dramatically between July 2002 and March 2003 with an exceptionally dry period between December 2002 and February 2003. Precipitation then increased again during Spring 2003, although rainfall levels were not as great as those observed in Spring 2002.

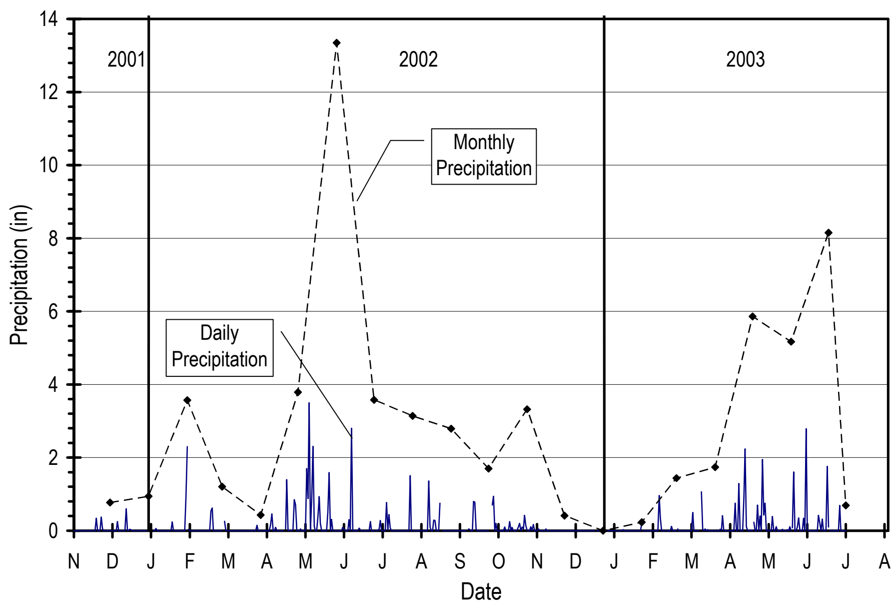


Figure 5.31 Monthly and daily precipitation from Lee's Summit Municipal Airport located approximately 4 miles east of I435-Kansas City sites.

5.5.2. Performance of I435-Wornall Road Site

Piezometers and Moisture Sensors. Figure 5.32 shows the water levels measured in the piezometers placed in the I435-Wornall Road slope. The first two readings taken are somewhat sporadic and are believed to be a result of the piezometers coming to equilibrium with the surrounding soils rather than a result of the actual water conditions within the slope. After this initial equalization period however, the piezometers appear to be providing reasonable readings. Piezometers P-1 and P-3, which are screened within the lower compacted clay shale, both consistently indicate decreasing water levels during the period October 2002 to March 2003 followed by increasing water levels since that time in response to increased precipitation (Figure 5.31). Water levels in these piezometers ranged from 5- to

8-ft (1.5- to 2.4-m) below grade. Piezometer P-2, located near the crest of the former slide area and screened within the upper stratum, shows a similar response to piezometers P-1 and P-3 with decreasing water levels during dry periods and increasing water levels during wet periods. However, the water levels in P-2 are significantly higher than those in P-1 and P-3, which suggests that a perched water condition develops within the upper stratum. Piezometer P-4, located near the toe of the slope within the upper stratum, had the highest water levels throughout the period of monitoring and the water level does not appear to respond to the observed precipitation. This suggests that the perched condition is maintained near the toe of the slope even during relatively dry periods.

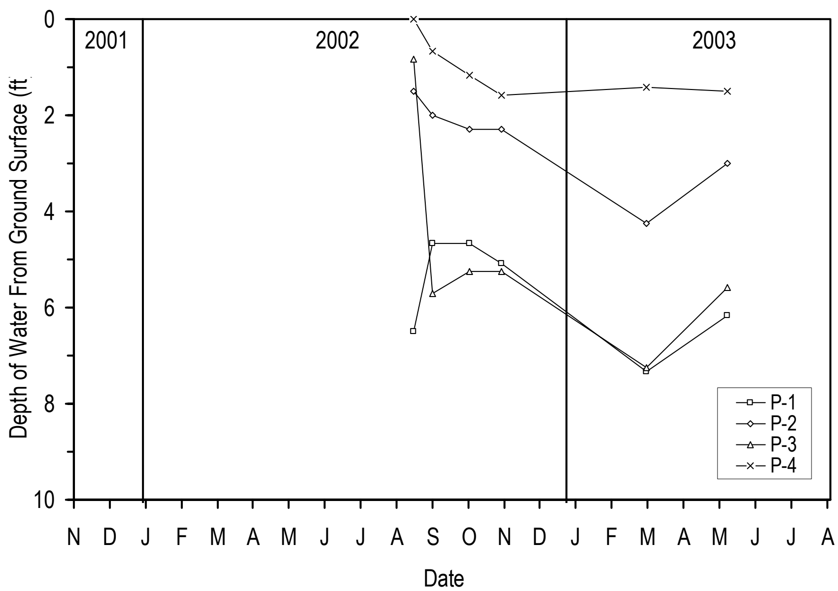


Figure 5.32 Piezometric water levels measured at I435-Wornall Road test site.

Readings taken for the Equitensiometers®, which measure soil suction directly, have consistently been out of range indicating that the soil has been essentially saturated since the sensors were installed. This further supports the piezometer readings for the shallow piezometers which suggest that positive pore pressures are present within the upper stratum. Readings taken for the ThetaProbes®, which measure volumetric water content, are plotted in Figure 5.33. Readings for both ThetaProbes® indicate increasing water content during Fall 2002. Water contents for the upper ThetaProbe® then decreased between November 2002 and February 2003 while water contents for the lower ThetaProbe® remained essentially constant during this time. Both probes indicate a significant increase in water content in early February 2003 followed by generally decreasing water contents since that time.

Inclinometers. Lateral deformations determined from inclinometers I-3 and I-4 are plotted in Figures 5.34 and 5.35, respectively. As shown in the figures, two different forms of deflection profiles have been observed in the inclinometers. Inclinometer I-3 indicates that maximum deformations are occurring near the ground surface with continuously decreasing deformations with depth. Inclinometer I-1 produced a similar profile with lower overall deformations. Inclinometer I-4 also shows that the maximum deformations are at the ground surface. However, I-4 also shows a significant discontinuity in the deformation profile at a depth between 8- and 10-ft (2.4- and 3.0-m), which may be an indication of the formation of a sliding surface. Inclinometer I-2 produced a similar profile with a discontinuity between 10- and 12-ft (3.0- and 3.6-m).

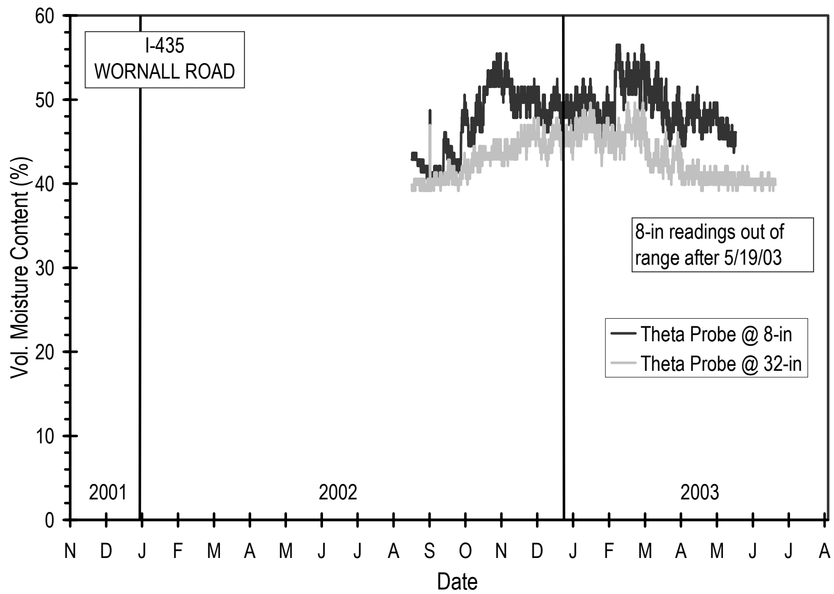


Figure 5.33 Volumetric water content from I435-Wornall Road test site.

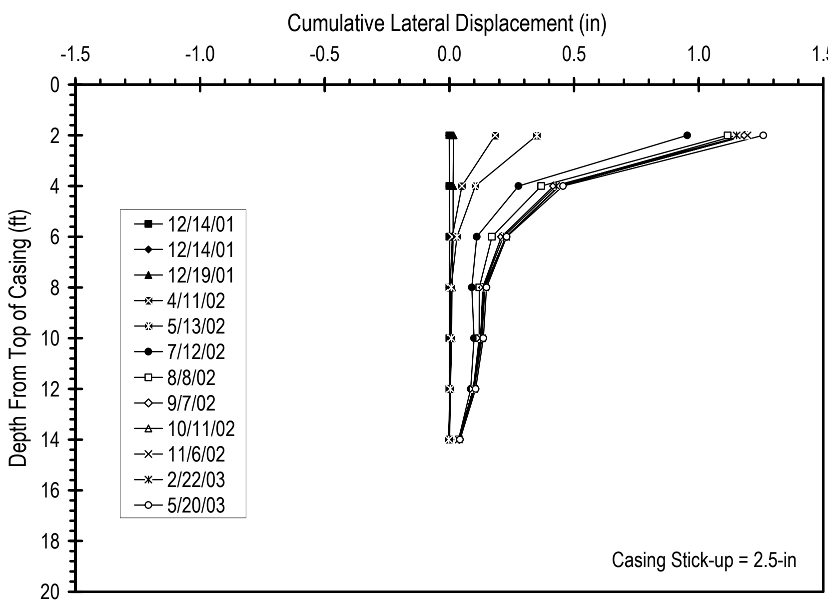


Figure 5.34 Lateral deflection profile for Inclinometer I-3 at I435-Wornall Road test site.

Cumulative deformations for all four inclinometers are plotted as a function of time in Figure 5.36. This figure shows that all four inclinometers have a similar trend of deformation with time. Little movement was observed during the first few months following installation. Movements then increased substantially in April 2002 and continued to increase throughout the summer until leveling out in August/September 2002. Movements since that time have generally been negligible. Maximum deformations for inclinometers I-2, I-3, and I-4 are approximately 1-in (2.5-cm) while deformations for inclinometer I-1 have been somewhat smaller.

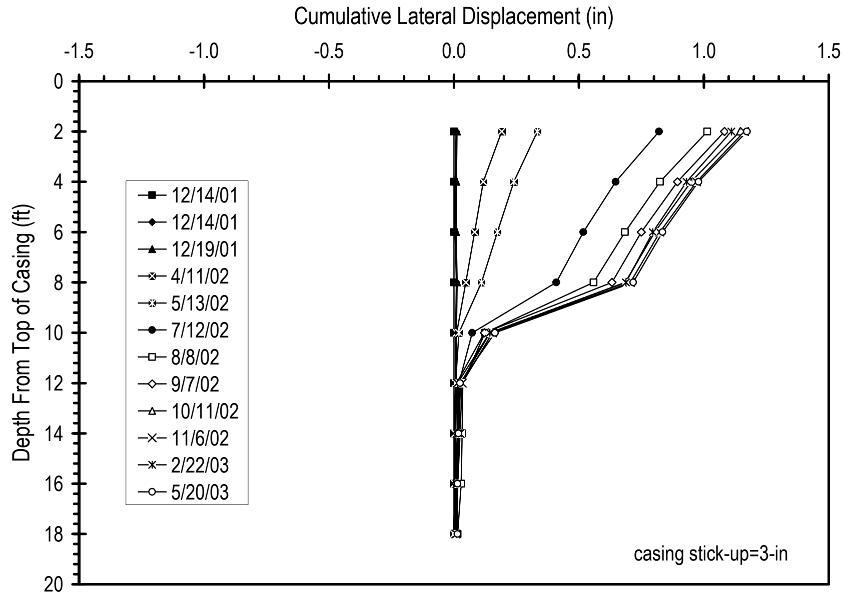


Figure 5.35 Lateral deflection profile for Inclinometer I-4 at I435-Wornall Road test site.

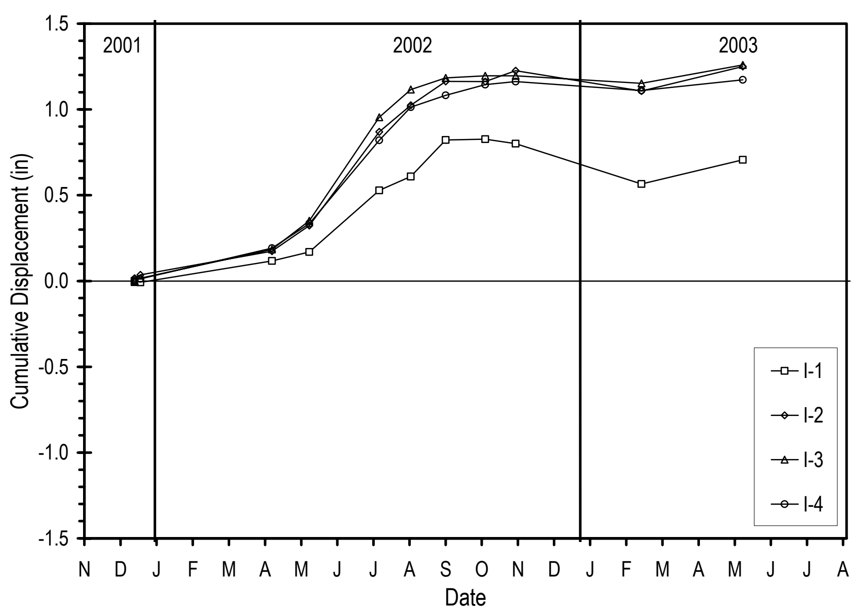


Figure 5.36 Cumulative lateral deflections for inclinometers I-1 through I-4 at the I435-Wornall Road test site.

The pattern of movements observed at the I435-Wornall Road slope is consistent with those observed at the I70-Emma site during Phase I, with an initial period of little movement followed by a period of increasing movements, after which movements are generally negligible. It is postulated that this movement sequence is a combined result of the pore pressure conditions within the slope and mobilization of resistance in the reinforcing members. Just after installation, the slope would be stable without the reinforcing members because of the low pore water pressures within the slope. However, during the first wet period following installation, the stability of the slope decreases in response to increasing pore water pressures. As the stability decreases, the slope begins to move at which point the reinforcing members begin to deflect and provide some resisting loads. With continued higher pore pressures, the slope movement continues until the reinforcing members mobilize loads sufficient to create equilibrium in the slope. At this point, additional movement is resisted by the reinforcing members and movement essentially stops. Upon subsequent wetting and drying cycles, some resistance in the members is already mobilized which prevents significant additional movement unless the pore water pressures become significantly greater than have been experienced since installation. In cases where subsequent pore pressures are greater than previously experienced, additional movement would mobilize the additional resistance required to reach a new equilibrium condition as long as no limit state is reached to produce failure. This hypothesis is supported by the movements shown in Figure 5.36 which shows very little movement since September 2002, despite the slope having been subjected to a relatively wet period between April and June 2003.

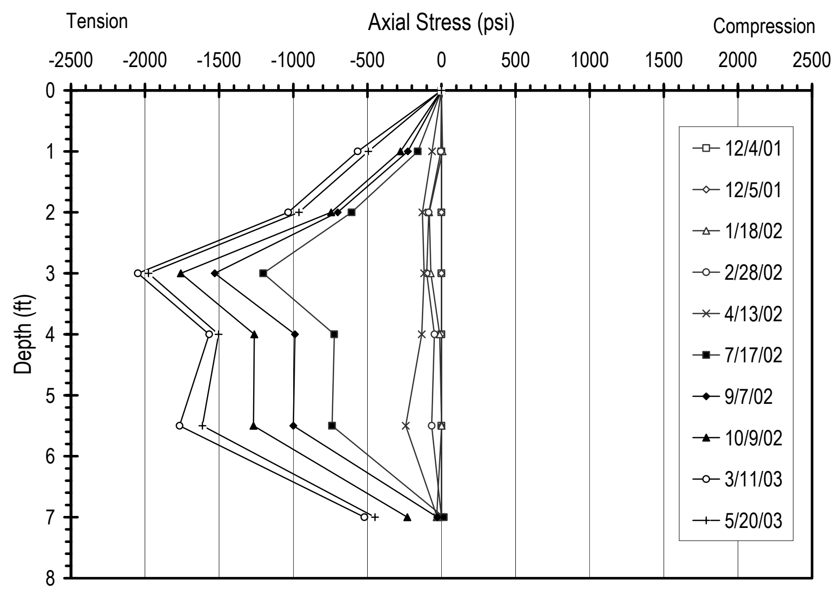
Instrumented Reinforcing Members. Readings from instrumented reinforcing members have also been taken on a regular basis. These readings have been processed and interpreted to establish the magnitudes of axial stresses and bending moments mobilized in the reinforcing members at each reading. However, it is important to note that reduction of the strain gage data requires that potentially significant assumptions be made in order to interpret the data. Two basic assumptions that were made include: (1) bending was assumed to be uniform when separating out axial strains from bending strains and (2) all strains were assumed to produce changes in stress (i.e. no creep or thermal strains). While these assumptions may be questioned, it is not clear that other assumptions could be made to reduce the data with the information currently available. Furthermore, there is no compelling evidence to suggest that these assumptions will have a noticeable impact on the interpretations made. Of somewhat more importance in the current context, however, are assumptions made regarding which gages were providing accurate data and which gages were not. To address this issue, several different interpretations have been developed for many of the instrumented reinforcing members (generally denoted as interpretation A, B, C, etc.) and significant effort has been put into selecting the most appropriate of these interpretations that is both reasonable and consistent with observations from other instrumentation. The interpretations presented in this and subsequent chapters are the ones deemed to be the most appropriate among several different interpretations that can be made.

One particular issue that came to light during interpretation of data from the instrumented reinforcing members is the issue of initial stresses and bending moments imposed during installation. Data obtained from members where readings were taken prior to and just after installation indicate that significant initial stresses and moments were often developed in the members due to the installation process. The existence of such stresses is not difficult to accept given the method of installation. However, the distribution of such stresses is in no way connected to the mechanisms by which load is transferred to the reinforcing members due to slope movements. It is therefore unreasonable to expect that the *overall* stresses and moments determined *including the initial stresses* should have distributions that are consistent with what one would expect from slope movements. However, it is reasonable to expect that the *incremental* stresses imposed *since installation* should have distributions that are consistent with those expected from slope movements. Because both the overall and incremental stresses are of importance in establishing the patterns of behavior in the slopes, two sets of interpretations were made: one set including any initial strains/stresses developed during installation and another that only includes the strains/stresses developed since installation was complete. In the following sections and chapters, the term "overall" is used to refer to stresses and bending moments determined to include any stresses or moments developed during installation (i.e. with reference to the unstressed member prior to installation) while the term "incremental" is used to refer to stresses and bending moments developed since installation (i.e. with reference to the member stresses just after installation).

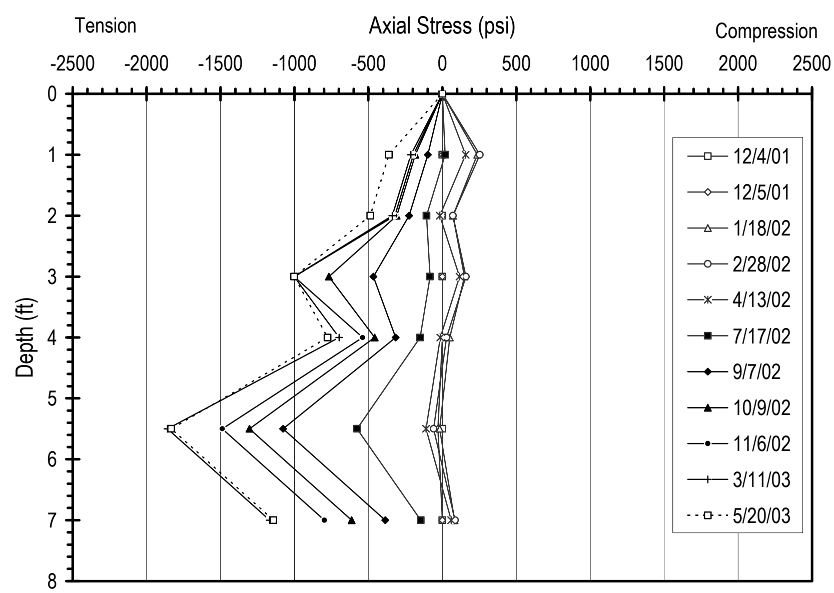
Figure 5.37 shows typical distributions of incremental axial stresses determined for the instrumented reinforcing members at the I435-Wornall Road test site. Members IM-1 and IM-2 had distributions of incremental axial stresses like that shown in Figure 5.37a, in which the maximum incremental axial stress is located near the midpoint of the member. In contrast, members IM-3 and IM-4 had distributions like the one shown in Figure 5.37b, with the maximum incremental stresses occurring at or near the tip of the members. Readings from all four instrumented members indicate development of tensile stresses since installation.

The maximum incremental and overall axial stresses in each member are plotted as a function of time in Figure 5.38. This figure shows that all four members had similar responses consisting of an initial period with little change in stress, followed by decreases in stress for a period of time, after which the axial stresses are essentially constant. The maximum incremental axial stresses for the four instrumented members ranged from approximately -1000- to -2000-psi (-6900- to -14,800-kPa). These stresses are much greater than those measured at other field sites as described in subsequent chapters. The initial axial stresses developed in the members during installation were small (<200 psi).

Results from instrumentation readings indicate that all four members have experienced tensile strains/stresses since installation as shown in Figure 5.38a. The reason(s) for the development of tensile strains/stresses in the members following installation is not entirely understood, but the trend has been consistently observed at all field test sites. One explanation currently being considered is that the observed tensile strains/stresses are a result of relaxation of compressive stresses induced in the members during installation. However, if this were true, it would suggest that the overall magnitude of the axial stresses would remain compressive or near zero, but would not become significantly negative since there is no apparent loading mechanism to induce tension. However, the data clearly indicate significant overall tensile strains/stresses, which does not support this hypothesis. It is possible that the strain gages may not have accurately captured the full magnitude of the strains induced during installation, which would suggest that the actual overall stresses (including installation induced stresses) were actually shifted by some unknown amount. Another possible contributor to the tensile strains/stresses could be a result of thermal strains produced by changes in temperature. However, there is no apparent trend to the strains according to season so this is not believed to be a major contributor to the tensile strains. Additional interpretation efforts will be made in the future in an attempt to resolve this issue.



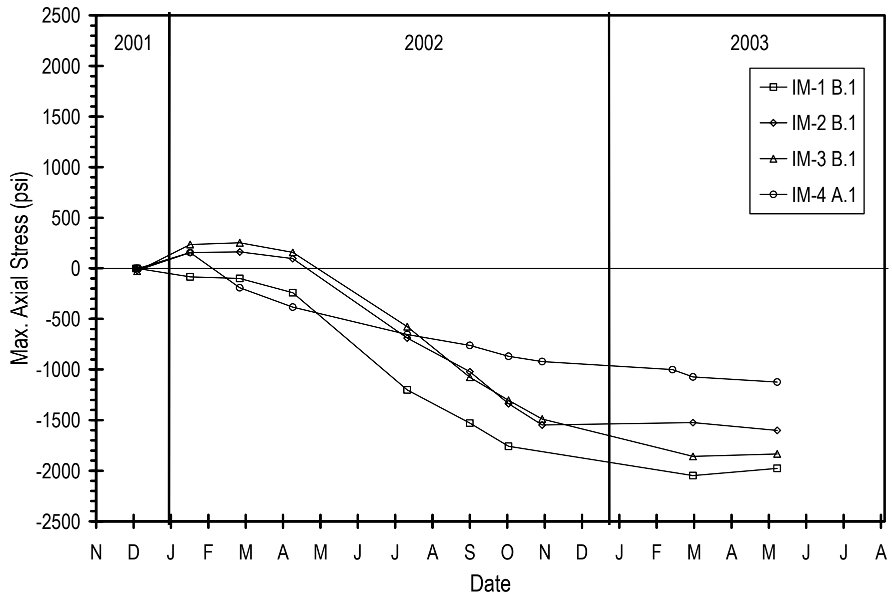
(a) instrumented member IM-1



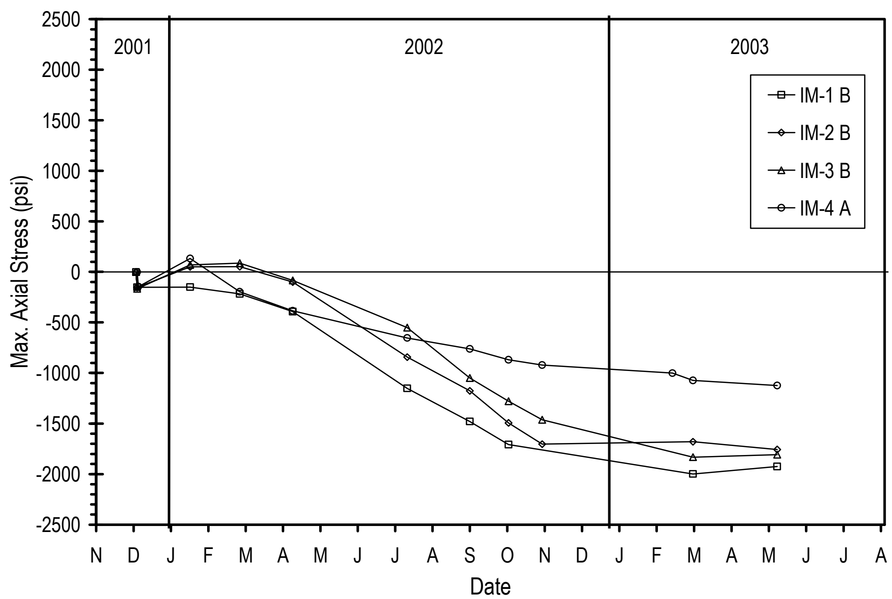
(b) instrumented member IM-3

Figure 5.37 Measured incremental axial stresses in instrumented members at I435-Wornall Road test site: (a) IM-1 and (b) IM-3.

Figures 5.39 through 5.41 show the incremental distributions of bending moments determined for instrumented members IM-1, IM-2, and IM-4, respectively. The distributions of bending moments for these three members are distinctly different. Member IM-1 produced a roughly parabolic distribution of bending moments with depth, with all moments being negative (implying bending towards the crest of the slope). Member IM-2 also produced a roughly parabolic distribution of bending moments, but with all moments being positive (implying bending towards the toe of the slope). One would expect that the bending moments in these two members would be very similar given their close proximity on the slope so the difference in the sign of the moments is perplexing. Member IM-3, located near the toe of the slope produced a distribution similar to IM-2. Member IM-4 has a somewhat different distribution of moments, with positive moments near the top of the member, near zero moments near the center of the member and positive moments in the lower portion of the member.



(a) incremental axial stress



(b) overall axial stress

Figure 5.38 Maximum axial stress in instrumented members at I435-Wornall Road test site: (a) incremental axial stresses, and (b) overall axial stresses.

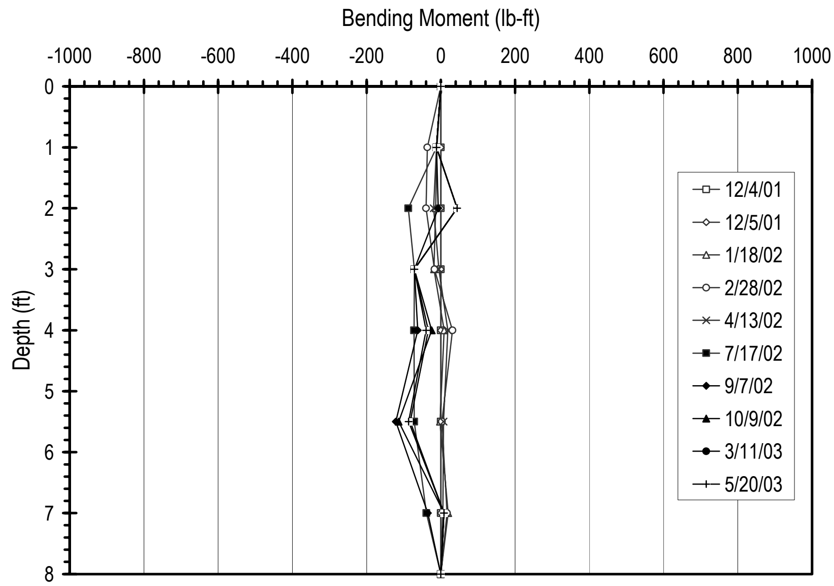


Figure 5.39 Measured incremental bending moments in instrumented member IM-1 at the I435-Wornall Road test site.

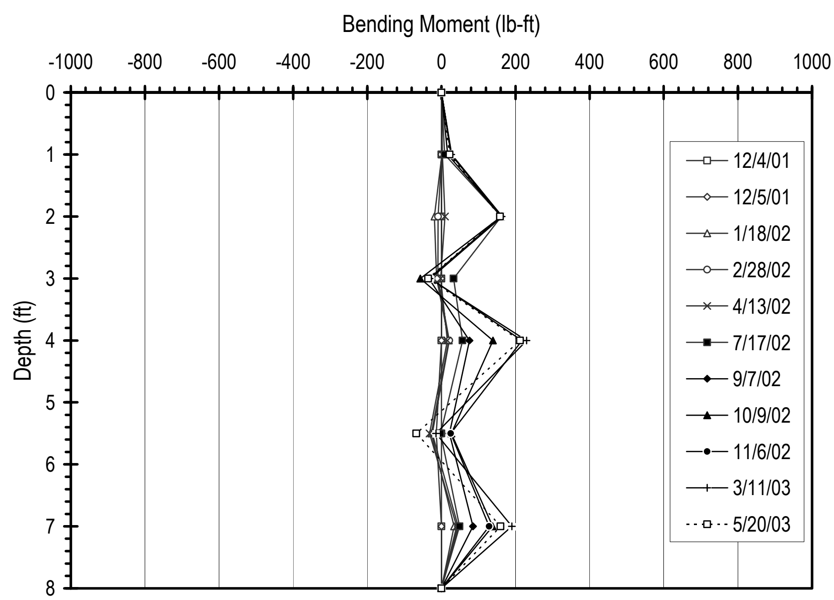


Figure 5.40 Measured incremental bending moments in instrumented member IM-2 at the I435-Wornall Road test site.

Figure 5.42 shows the maximum bending moments determined for each instrumented reinforcing member plotted as a function of time. As shown in Figure 5.42b, the initial bending moments developed during installation generally ranged from 20- to 120-lb-ft (27-160-N-m). Incremental bending moments in each member remained relatively low for several months followed by a period of relatively steady increases in the bending moments during and just following the period of high precipitation between April and June 2002.

Since that time the maximum bending moments have remained steady, even during the spring of 2003. Member IM-3, located near the toe of the slope where pore pressures have been highest, has experienced the largest incremental and overall bending moments. However, these moments remain below 500-lb-ft (680-N-m), which indicates that the members have significant excess capacity remaining and are not near failure (nominal capacity is 1000-lb-ft).

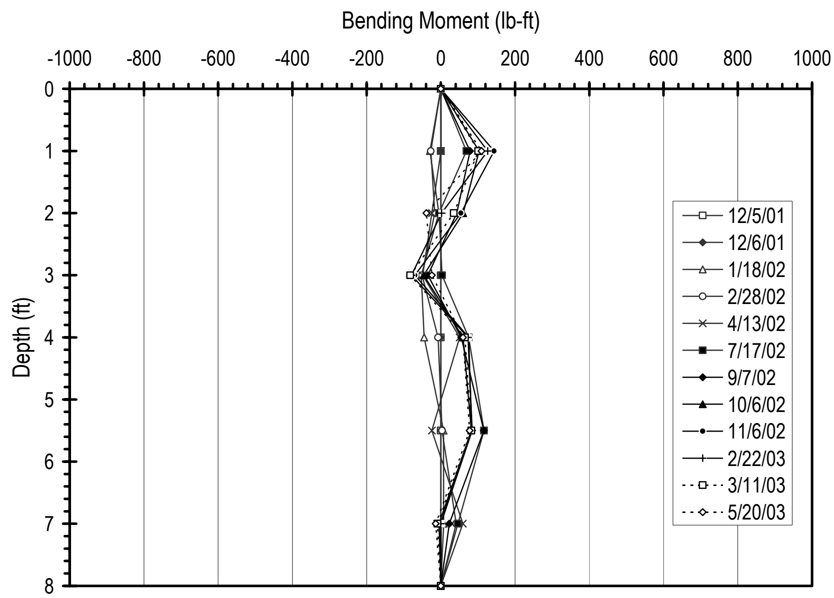


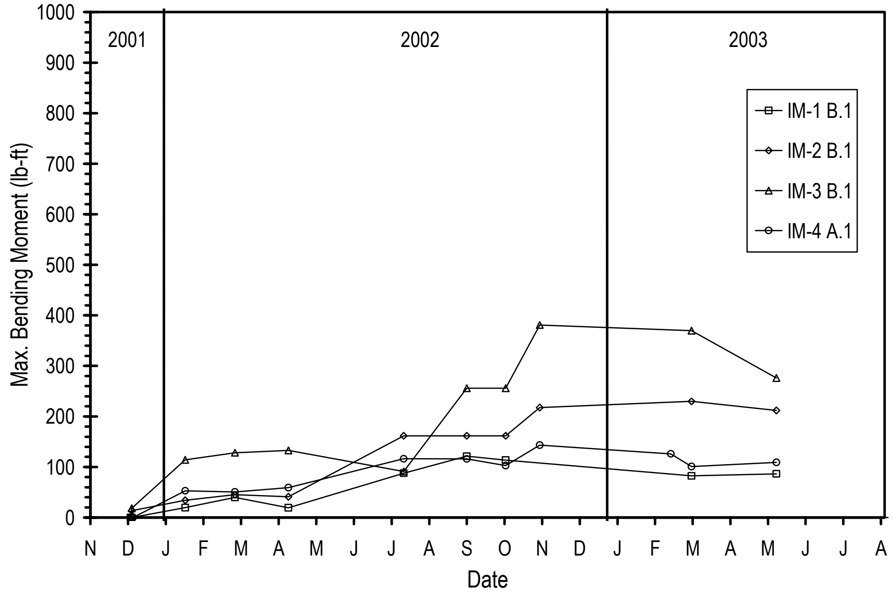
Figure 5.41 Measured incremental bending moments in instrumented member IM-4 at the I435-Wornall Road test site.

To date, readings taken from the FSR on the instrumented reinforcing members have been below the "detection limit" of 1000-psf (50-kPa) discussed previously. While this limits the information provided by these sensors to some extent, these readings at least serve as an upper limit on the magnitude of the lateral pressures being applied to the reinforcing members, which is useful when interpreting the data from the strain gages. It is also hoped that the FSR will provide more quantitative information regarding the lateral pressures if the slope reaches a condition whereby the soil fails around the reinforcing members.

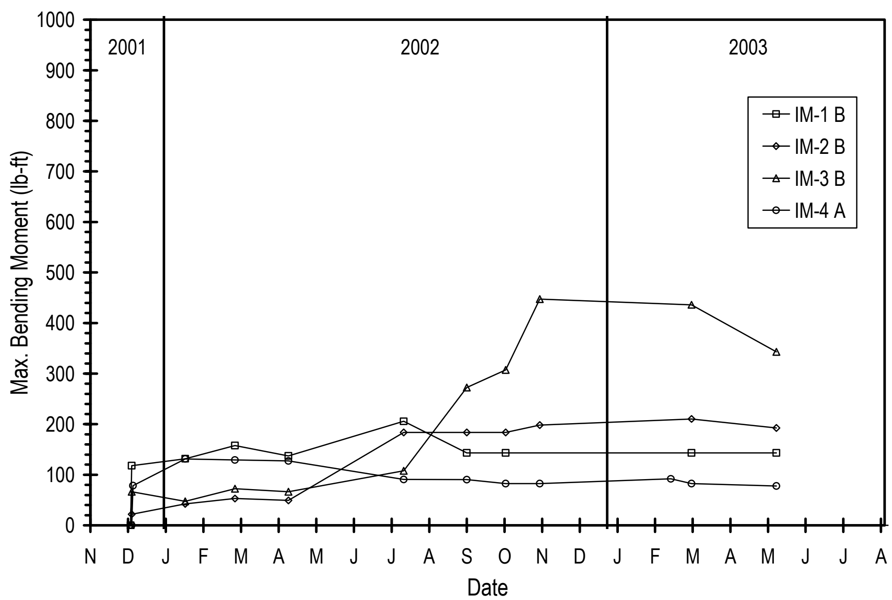
Overall, the performance observed with the different types of instrumentation show a consistent behavioral pattern. Slope movements and mobilized loads in the reinforcing members increased in a consistent manner during the first period where pore water pressures were observed to increase following installation. Since that time, both the loads in the reinforcing members and the deformations observed in the slope have remained essentially constant. The fact that both the loads and deformations remained constant during the relatively wet period experienced in Spring 2003 indicates that the loads mobilized during the previous year were sufficient to maintain the equilibrium of the slope.

5.5.3. Performance of I435-Holmes Road and I435-Control Sites

Measured piezometric water levels recorded at the I435-Holmes Road site are plotted in Figure 5.43. These data again indicate the presence of a perched water condition within the slope. The piezometric levels seem to be somewhat less responsive to rainfall events than those measured at the I435-Wornall Road site, particularly during Spring 2003. Figures 5.44 and 5.45 show the lateral deformations determined at the I435-Holmes Road and I435-Control sites, respectively. To date, neither inclinometer has experienced any significant movement. Measured incremental axial stresses and bending moments in instrumented members in the I435-Holmes Road slope have also been negligible.



(a) incremental bending moments



(b) overall bending moments

Figure 5.42 Maximum bending moments in instrumented members at I435-Wornall Road test site: (a) incremental bending moments and (b) overall bending moments.

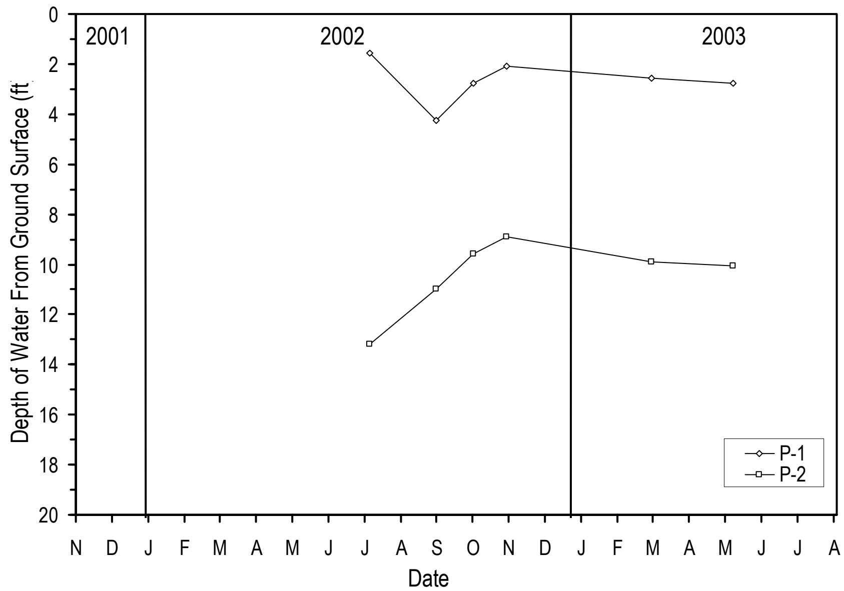


Figure 5.43 Piezometric water levels measured at I435-Holmes Road test site.

5.6. Status of Instrumentation at I435-Kansas City sites

The instrumentation installed at the I435-Kansas City sites has provided valuable information to help develop a more thorough understanding of the stability of the stabilized slopes and the load transfer mechanisms by which load is transferred to the reinforcing members. Much of the instrumentation installed is still in place and functional. However, some of the instrumentation has deteriorated as described below.

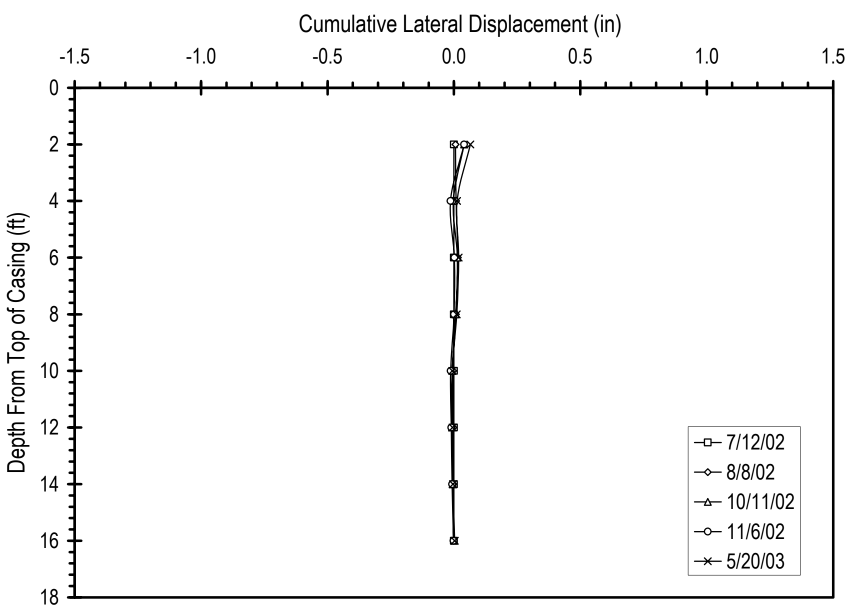


Figure 5.44 Lateral deflection profile for Inclinometer I-1 at I435-Holmes Road test site.

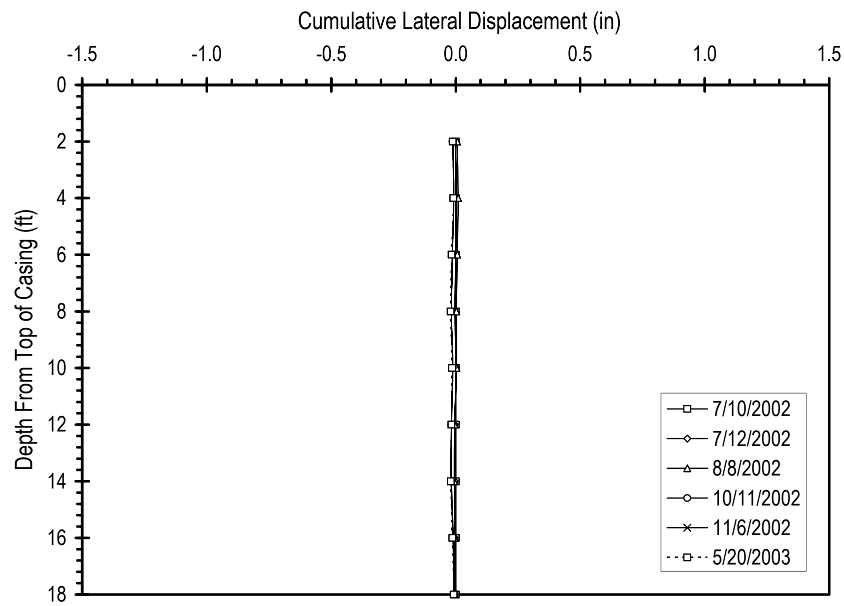


Figure 5.45 Lateral deflection profile for Inclinometer I-1 at I435 control slide.

The inclinometers and piezometers continue to perform well and provide valuable information. Both of these types of instruments are extremely robust and expected to continue providing valuable data for the foreseeable future.

The moisture instrumentation has also performed reasonably well. The ThetaProbes® installed near the center of the slide area have provided confirmation of changes in soil moisture during the monitoring period and the essentially continuous nature of the readings is helpful for interpolating between discrete readings of the standpipe piezometers. Laboratory testing currently underway to establish appropriate soil-water characteristic curves for the I435 soils will also permit these moisture contents to be used to estimate pore pressures in the slope. Readings taken for the Equitensiometers® have been out of range throughout the duration of monitoring. While this has limited the value of these sensors to some extent, these readings have confirmed that positive pore water pressures are present within the surficial soils. Since the slope appears to have positive pore pressure conditions throughout most of the year, some consideration is being given to installation of alternative sensors that can measure positive pore water pressures on a continuous basis to further facilitate our understanding of the pore pressure conditions at the site. Readings continue to be taken with the Profile Probe® and access tubes distributed across the site. Readings taken to date have been extremely scattered so serious consideration will be given to the significance and accuracy of these measurements.

The instrumented reinforcing members are in poorer condition and have not performed as well as similar members installed at the I70-Emma test site during Phase I. At present, approximately 75 percent of the strain gages mounted on the instrumented reinforcing members have lost functionality, which makes interpretation of subsequent readings almost impossible. The reasons for the poor performance are not known, but it is speculated that many of the gages have been damaged by moisture. The instrumented recycled plastic members installed at the I435-Wornall Road site had a higher filler content (primarily sawdust) than previous instrumented members, which may have acted as a conduit for water and possibly resulted in a number of the strain gages becoming delaminated from the reinforcing members. The I435-Wornall Road slope also appears to be much wetter than the I70-Emma site in general which has further compounded the moisture problems. We are continuing to work on ways to better evaluate the data from the remaining gages, in addition to developing means for estimating bending moments from inclinometer measurements to provide estimates of the bending moments in the reinforcing members as more time passes.

The FSR appear to continue to be functional. However, they will not begin to provide quantitative data until the lateral pressures exceed the detection limit. Even if the mobilized lateral pressures remain below the detection limit of the sensors, the limit will serve as an upper bound of the mobilized lateral pressures, which will facilitate further interpretation of the instrumentation measurements.

5.7. Summary

In this chapter, the activities undertaken to establish and monitor two stabilized test sections and one control section at the I435-Kansas City sites have been presented. The general characteristics of the three slide areas were described including results from a site investigation and laboratory testing program. An extensive series of stability analyses performed to estimate the stability of the stabilized slopes was then presented and the selected stabilization schemes were described. Activities undertaken to install reinforcing members in the two stabilized slopes were then described. The different types of instrumentation used at the I435-Kansas City site and other future sites were also described. Finally, results obtained from monitoring the field instrumentation over the past twenty months were presented and the status of the instrumentation at the site described.

Chapter 6. US36-Stewartsville Site

The second site stabilized during Phase II of the project is the US36-Stewartsville site. The site is located in northwest Missouri on U.S. Highway 36, approximately two miles west of the city of Stewartsville. In this chapter, the general characteristics of the site are first described followed by descriptions of the design analyses performed to select the stabilization schemes used at the site and the selected stabilization schemes. Field installation activities at the site are then summarized. Finally, the instrumentation scheme used to monitor the performance of the site and the results of instrumentation readings to date are presented.

6.1. Site Characteristics

The slope stabilized at the US36-Stewartsville site lies in the median of US36 between the eastbound and westbound sections of the roadway. Figure 6.1 shows an air photo of the site indicating the location of the slope and Figure 6.2 shows a photo of the site following the recent slide event which involved approximately 150-ft (45-m) of the slope (measured parallel to US36). The slope at the site is approximately 29-ft (8.8-m) high with an inclination of 2.2H:1V. The slope is similar to the slopes at the I435-Kansas City sites in that the stratigraphy consists of a surficial layer of soft to medium clay overlying stiff to hard fat clay. However, the slope is an excavated slope rather than an embankment fill. A second, much smaller slide area is located approximately 100-ft (30-m) to the west of the main slide area. This slide area was selected for use as a control slope for the main slide.

Figure 6.1 Air photo of US36-Stewartsville site taken March 26, 1997 showing location of site in the median of US36 (from USGS).

Boring and sampling at the US36-Stewartsville site was performed by MoDOT Soils and Geology crews during the period May 30 to June 7, 2001. A plan view of the site showing the locations of all borings is provided in Appendix B along with all boring logs. A total of eight 4-in (10-cm) diameter solid stem auger borings were made to depths varying from 10-ft (3-m) to 25-ft (7.5-m). In seven of the borings, continuous 3-in (7.6-cm) diameter Shelby tube samples were taken for field classification and laboratory testing in a manner similar to that described in Chapter 5. The general stratigraphy determined from these borings consists of a 2.5- to 5-ft (0.8- to 1.5-m) thick stratum of soft, moist lean to fat clay overlying very stiff to hard fat clay with scattered gravel to the depths investigated. In the remaining boring, "continuous" Standard Penetration tests (SPT) were performed. SPT N_{60}-values determined from tests in the upper 3.0-ft (0.9-m) of the boring were 0 (weight of hammer). Between 3- and 6-ft (0.9- and 1.8-m), N_{60} ranged from 9 to 11. Below 6-ft (1.8-m), N_{60} increased dramatically and varied between 13 and 20. N_{60} values determined for SPT tests performed in other borings when Shelby tube samples could not be taken showed similarly high SPT N-values within the lower stiff clay. No groundwater was observed in any of the boreholes during the site investigation.

Figure 6.2 Photograph of US36-Stewartsville site taken after the recent slide at the site.

Laboratory testing performed on samples taken from the site included natural moisture contents, Atterberg limits, and triaxial compression tests. Moisture contents varied somewhat across the site, but most borings indicated higher moisture contents in the upper 5- to 10-ft (1.5- to 3.0-m) of the profile below which the moisture content generally decreased to essentially constant values of approximately 20 percent in all borings. Moisture contents in the surficial soils ranged from 18 to 44 percent and averaged about 30 percent.

Atterberg limits for the surficial soils varied substantially. Liquid limits (LL) for the surficial soils ranged from 33 to 69 and plastic limits (PL) varied from 16 to 26; plasticity indices (PI) for these soils ranged from 7 to 44. Most of the surficial samples tested classified as CL soils in the Unified Soil Classification System (USCS), although several samples classified as CH and one sample classified as ML. Atterberg limits for the deeper soils were more consistent with LL ranging from 41 to 55, PL ranging from 16 to 21, and PI ranging from 21 to 34. All of the deeper soils classified as CL or CH in the USCS.

Consolidated-undrained ($\overline{\text{CU}}$, $\overline{R}$) and consolidated-drained (CD, S) type triaxial compression tests were performed on a total of 13 specimens from the US36-Stewartsville site. Figure 6.3 shows the stress paths determined from these tests along with "upper bound" and "lower bound" failure envelopes for specimens taken from depths less than 6-ft (1.8-m) and depths greater than 6-ft (1.8-m), respectively. Mohr-Coulomb effective stress strength parameters for these envelopes are summarized in Table 6.1. Tests on specimens from shallow depths indicate the soil has a small cohesion intercept ($\overline{c}$) and an angle of internal friction ($\overline{\phi}$) between 27° and 29°. Tests on deeper specimens indicate that $\overline{c}$ is between 0- and 211-psf (10.1-kPa) and $\overline{\phi}$ is between 32° and 35°.

Table 6.1 Summary of Mohr-Coulomb effective stress strength parameters from triaxial compression tests on specimens from the US36-Stewartsville test site.

			Upper bound		Lower bound	
Stratum	Depths	Sample Numbers	$\overline{c}$ (psf)	$\overline{\phi}$ (degrees)	$\overline{c}$ (psf)	$\overline{\phi}$ (degrees)
Surficial lean to fat clay	< 6.0-ft	276, 278, 280, 316, 318, 343	42	29	0	27
Deeper stiff clay	> 6-ft	282, 300, 302, 322, 324, 375	211	35	0	32.5

6.2. Design of Stabilization Schemes

An extensive series of stability analyses was performed to select the stabilization schemes to be used at the US36-Stewartsville site. The design cross-section utilized for these analyses is shown in Figure 6.4. The ground surface profile was established from survey data provided by MoDOT and the subsurface geometry was assumed to consist of a 3- to 5-ft (0.9- to 1.5-m) thick, soft surficial layer overlying a layer with much higher strength based on boring logs obtained for the site.

The approach used for the stability analyses was similar to that used for the I435-Wornall Road site wherein a series of back-analyses was performed to establish a range of plausible conditions that could have led to the failure, followed by additional analyses for a variety of different reinforcement configurations to establish factors of safety for possible reinforcement schemes. The range of slope conditions evaluated included cases with zero pore pressures throughout the slope and a perched water condition within the upper stratum for various assumed thicknesses of the upper stratum. Analyses were performed for both the upper and lower bound strength parameters as well as for several other assumed sets of strength conditions. Based on these analyses, the stability cases summarized in Table 6.2 were selected as plausible conditions that could have led to failure of the slope.

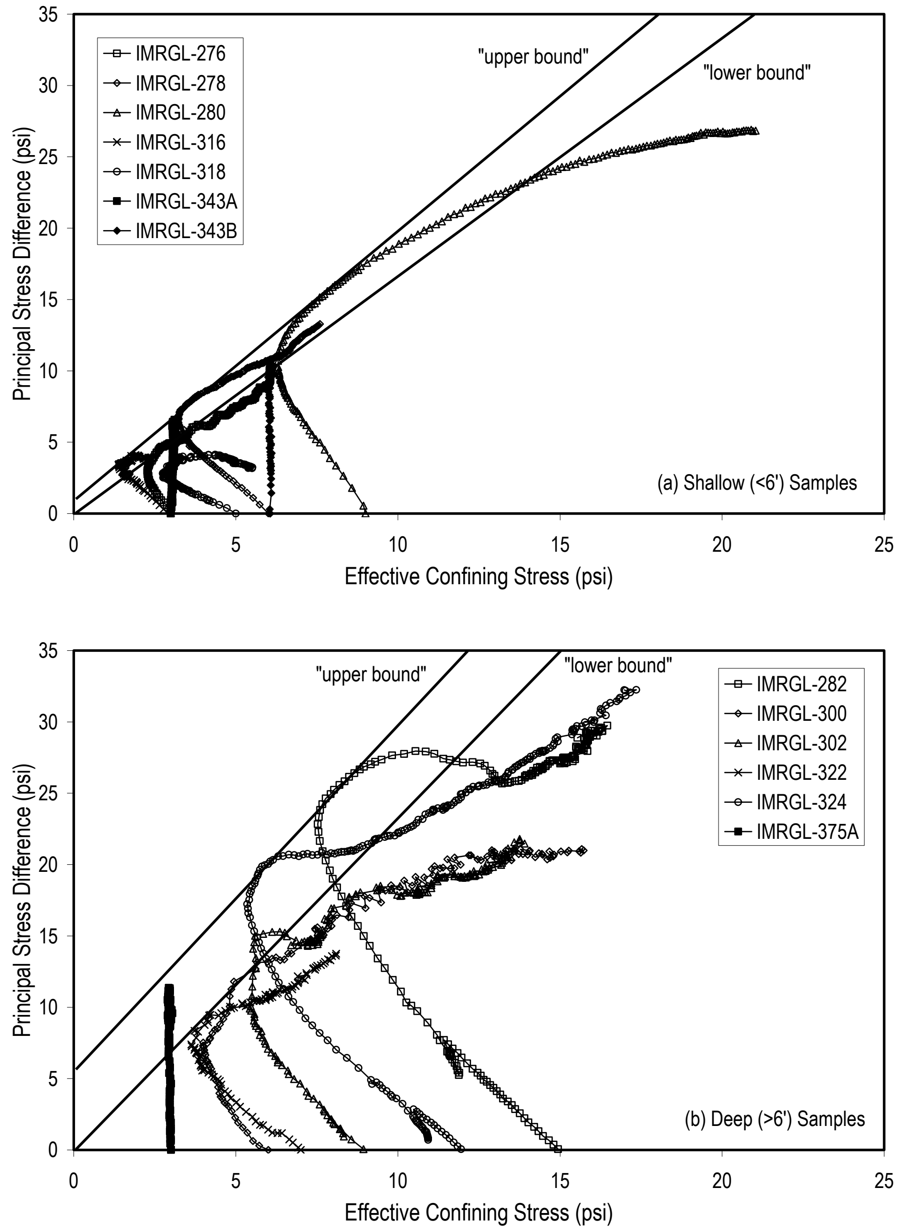


Figure 6.3 Summary of triaxial test results for specimens from US36-Stewartsville site: (a) shallow samples and (b) deeper samples.

Several reinforcement configurations were then evaluated for each of the plausible stability cases. The reinforcement configurations included reinforcing members placed in a uniform grid over the entire slope face with members spaced between 3- and 6-ft (0.9- and 1.8-m) as well as several non-uniform grids with different member configurations in the upper, middle, and lower portions of the slope. Table 6.3 shows a summary of the different configurations analyzed and the resulting factors of safety for the different plausible stability cases considered. The calculated factors of safety ranged from a low of 1.03 for the most widely spaced members considered to 1.30 for the most closely spaced members. In general, factors of safety calculated for the different member configurations using the upper and lower bound envelopes for the lower stiff clay were identical, which indicates that the critical sliding surface passes only through the upper stratum in all cases.

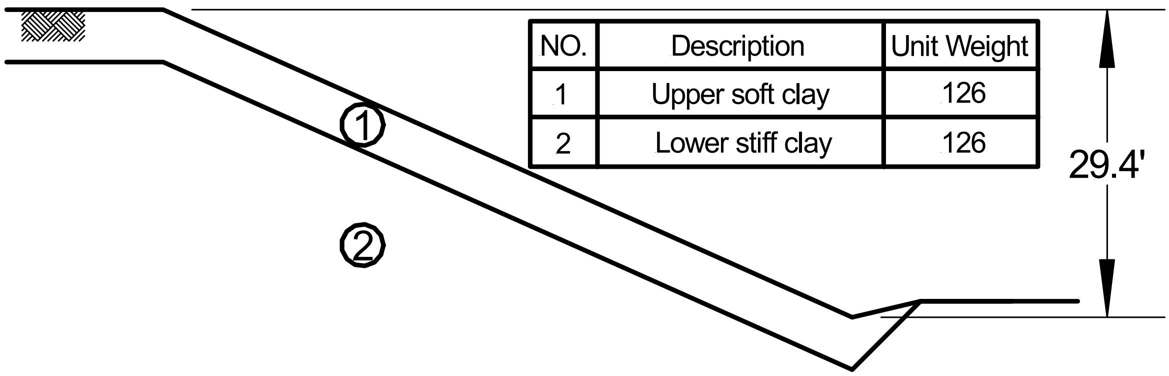


Figure 6.4 Design cross-section for the US36-Stewartsville slope.

Table 6.2 Summary of plausible stability cases leading to the failure at the US36-Stewartsville test site.

Stability Case	Thickness of Upper Stratum (ft)	Pore Pressure Condition	Deeper Stratum Envelope	Upper Stratum Assumed Parameter	Upper Stratum Back-calculated Parameter
A	3.0	perched 1 [1]	upper bound	$\bar{\phi}$ =29°	$\bar{c}$ =45 psf
B	5.0	perched 1	upper bound	$\bar{\phi}$ =28°	$\bar{c}$ =76 psf
C	3.0	perched 2 [2]	upper bound	$\bar{c}$ =1-psf	$\bar{\phi}$ =26°
D	5.0	perched 3 [3]	upper bound	$\bar{c}$ =1-psf	$\bar{\phi}$ =26°
E	3.0	perched 1	lower bound	$\bar{\phi}$ =29°	$\bar{c}$ =45 psf
F	5.0	perched 1	lower bound	$\bar{\phi}$ =28°	$\bar{c}$ =76 psf
G	3.0	perched 2	lower bound	$\bar{c}$ =1-psf	$\bar{\phi}$ =26°
H	5.0	perched 3	lower bound	$\bar{c}$ =1-psf	$\bar{\phi}$ =26°

[1] condition with piezometric surface for upper stratum at ground surface and u=0 for deeper stratum
[2] condition with piezometric surface for upper stratum 2-ft below ground surface and u=0 for deeper stratum
[3] condition with piezometric surface for upper stratum 3.3-ft below ground surface and u=0 for deeper stratum

Because the 3-ft by 3-ft (0.9-m by 0.9-m) arrays of reinforcing members used at both the I70-Emma and I435-Wornall Road sites seemed to be sufficient for stabilization of those slopes and because significant costs savings could be realized if more widely spaced arrays of reinforcing members could be shown to be effective, several different configurations of reinforcing members were selected for use at the US36-Stewartsville site. Using more widely spaced arrays of reinforcing members also increases the chances of having a failure at the site, which would greatly facilitate calibration of the design method described in Chapter 3. Figure 6.5 shows a plan view of the site with the selected reinforcement configurations. The slope was divided into four different sections, denoted Sections A through D, with a different configuration selected for each section. Section A had members placed on a 4.5-ft by 3.0-ft (1.4-m by 0.9-m) staggered grid, Section B a 6.0-ft by 6.0-ft (1.8-m by 1.8-m) staggered grid, Section C a 6.0-ft by 4.5-ft (1.8-m by 1.4-m) staggered grid, and Section D a

4.5-ft by 6.0-ft (1.4-m by 1.8-m) staggered grid. All members were to be installed with a vertical orientation. Estimated factors of safety for each of these reinforced sections are summarized in Table 6.4.

Table 6.3 Summary of factors of safety determined for different reinforcement configurations and stability cases for the US36-Stewartsville test site.

Rein. Spacing (ft)	Factor of Safety for Respective Stability Case							
	A	B	C	D	E	F	G	H
3L x 3T [1]	1.12	1.29	1.16	1.30	1.12	1.29	1.16	1.30
4L x 3T	1.08	1.19	1.11	1.20	1.08	1.19	1.11	1.20
3L x 6T	1.04	1.12	1.07	1.14	1.04	1.12	1.07	1.14
5L x 3T	1.07	1.13	1.08	1.15	1.07	1.13	1.08	1.15
5L x 6T	1.03	1.06	1.04	1.08	1.03	1.06	1.04	1.07
4L x 6T	1.04	1.06	1.05	1.09	1.04	1.06	1.05	1.09
3L x 3T and 3L x 6T [2]	1.12	1.19	--	--	--	--	--	--
3L x 3T and 3L x 6T [3]	1.11	1.17	--	--	--	--	--	--

[1] L and T denote spacing in longitudinal (strike) and transverse (dip) directions, respectively
[2] 3L x 3T grid in middle third of slope, 3L x 6T grid elsewhere
[3] 3L x 6T grid for upper and lower four rows of reinforcement, 3L x 3T grid elsewhere

6.3. Field Installation

The main slide area and control slide area were regraded to their original slopes in spring 2002. Installation of reinforcing members was performed during the period April 30 to May 7, 2002. The equipment utilized at the US36-Stewartsville site was the Ingersoll Rand (IR) CM150 rig that was previously utilized at the I435-Wornall Road and I435-Holmes Road sites (Figure 5.11b). Because of the steep slope and the fact that the guardrail for the westbound lanes was located approximately 20-ft (6-m) back from the crest of the slope, the rig was tethered to a truck located between the guardrail and the crest of the slope to help control the rig during driving. Otherwise, the rig performed well. Figure 6.6 shows a photograph of the site near the end of installation.

Table 6.4 Summary of estimated factors of safety for each reinforced slope section at the US36-Stewartsville test site.

Slope Section	Reinforcing Scheme	Estimated Factor of Safety
A	4.5L x 3.0T	1.07 – 1.20
B	6.0L x 6.0T	1.03 – 1.08
C	6.0L x 4.5T	1.03 – 1.15
D	4.5L x 6.0T	1.03 – 1.09

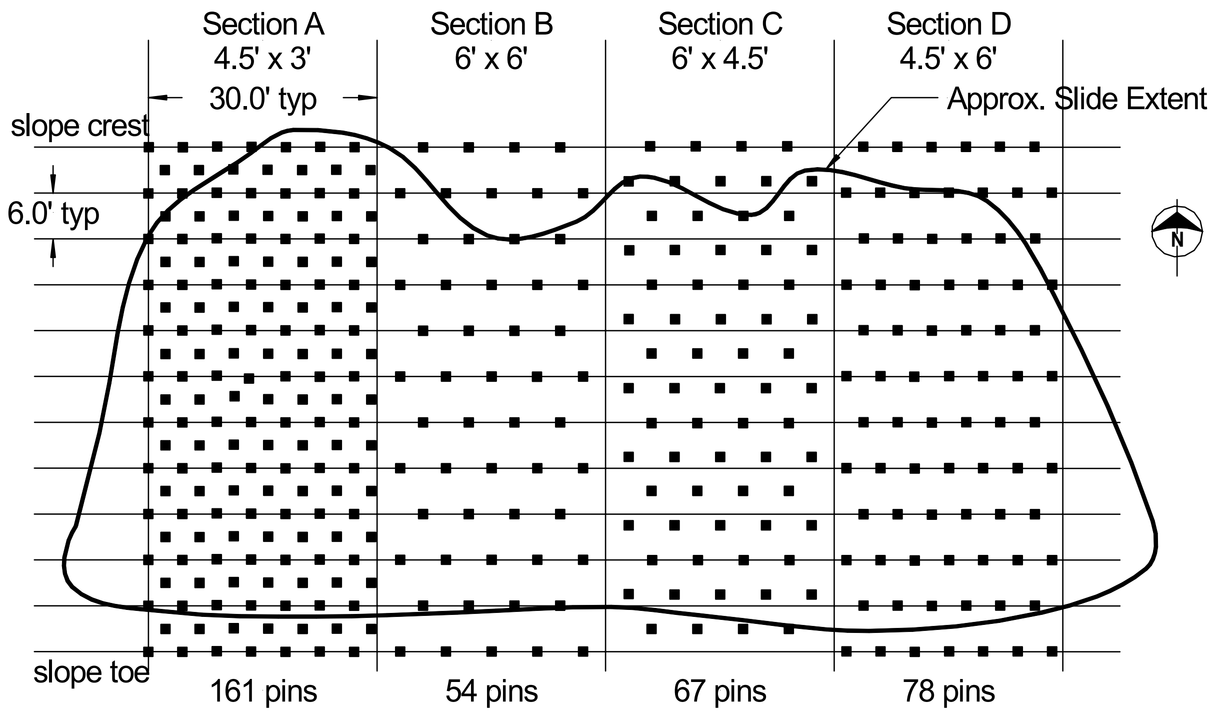


Figure 6.5 Plan view of selected stabilization schemes for US36-Stewartsville site (Note that all members were installed with vertical alignment).

Figure 6.6 Photograph of US36-Stewartsville site nearing the completion of installation.

A total of 360 recycled plastic members from Batch A6 were installed at the site. Only 59 of the members installed were driven to full depth; most of these were located near the crest of the slope. Members installed near the toe of the slope generally reached refusal at depths between 4- and 5-ft (1.2- and 1.5-m) while members installed further up the slope reached refusal at progressively greater depths. In cases where members could not be installed to full depth, effort was made to ensure that the members penetrated at least 6-in (15-cm) into the stiffer stratum to provide adequate anchorage. The portions of the members left above grade were then cut off using a chain saw.

One problem experienced during installation was that several members installed early in the installation sequence split apart along the mid-plane of the member and shattered once they had penetrated several feet into the ground. Inspection of the remaining members on site revealed small cracks on the ends of some members as shown in Figure 6.7 that were apparently developed during the manufacturing process. The entire stock of members was therefore inspected and all pallets containing members with cracks were returned to the manufacturer. Approximately 21 members from these pallets were installed along the easternmost portion of Section D prior to remedying the problem. However, no further problems were experienced with the remaining members and no significant defects were observed in subsequent batches of reinforcing members used for the remaining sites.

Figure 6.7 Photographs of defective recycled plastic members damaged during installation.

Average penetration rates were determined for 208 members installed at the site. Figure 6.8 shows a frequency distribution of the average penetration rates determined for these members. The mean of all penetration rates was 5.1-ft/min (1.5-m/min) with a standard deviation of 3.2-ft/min (1.0-m/min). Installation rates (including set up time) achieved at the US36-Stewartsville site were again relatively slow during the initial phases of installation, but increased significantly once the problem encountered with the defective members was addressed. The peak installation rate achieved at the US36-Stewartsville site was 93 members in one day, with an average rate of 70 members/day.

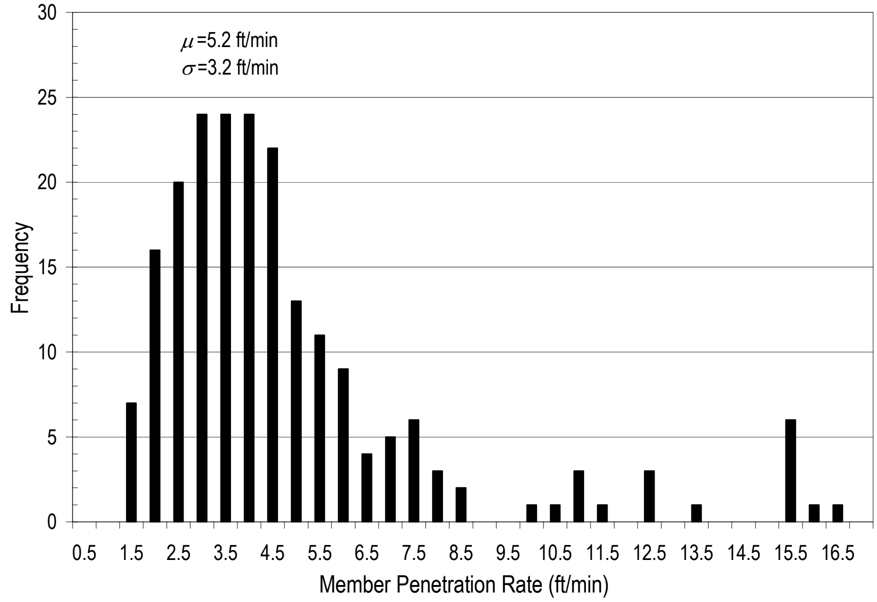


Figure 6.8 Frequency distribution of average penetration rates for recycled plastic members from Batch A5 installed at the US36-Stewartsville site. (μ=mean, σ=std. dev.)

Upon completion of installation, the slope was left unseeded for several weeks and heavy rains caused some minor erosion of the slope, which exposed a number of reinforcing members to depths of an inch or more. The exposed lengths were subsequently cut off as close to the ground surface as possible. Weeds and some grass grown from seed spread at the site have become somewhat established, which has helped to reduce erosion to some extent but minor erosion continues to occur during heavy rain events.

6.4. Instrumentation

Several different types of instrumentation were installed at the US36-Stewartsville site to monitor lateral deformations, moisture conditions, and loads in the reinforcing members. Figure 6.9 shows a schematic of the main slide area indicating approximate locations of the instrumentation installed. Additional instrumentation was also installed in the control section located to the west of the main slide as described below.

Five of the recycled plastic reinforcing members installed in the main slide area were instrumented with strain gages and force-sensing resistors (FSR) as described in Chapter 5. Table 6.5 summarizes the instrumented members installed and the "stick-up" length of the members remaining above ground after installation. Members IM-13 and IM-14 were installed in Section A while members IM-15, IM-11, and IM-12 were installed near the center of Sections B, C, and D, respectively. Two additional instrumented members were installed approximately 10-ft (3-m) apart near the center of the control slide as shown in Figure 6.10 to evaluate the "free-field" behavior of the reinforcing members. One of these members, member IM-7, is a recycled plastic member identical to the other instrumented recycled plastic members. The other member, member IM-9, is a 3.5-in (8.8-cm) diameter steel pipe instrumented with strain gages but no FSR as was done for the steel pipe at the I435-Holmes Road site.

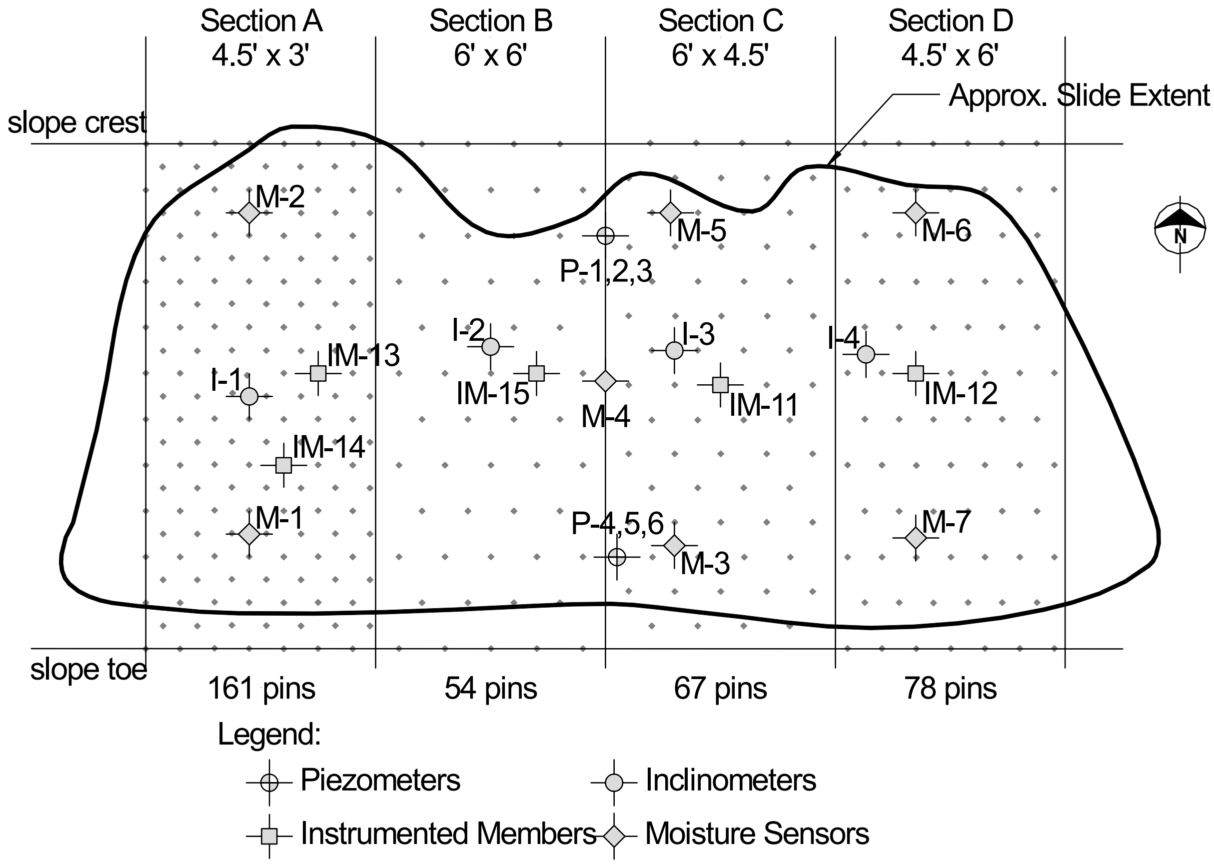


Figure 6.9 Instrumentation layout for US36-Stewartsville site.

Table 6.5 Summary of "stick-up" for instrumented reinforcing members at the US36-Stewartsville site.

Member Designation	Member Type	Slope Section	Stick-up Length (ft)
IM-11	plastic	C	2.5
IM-12	plastic	D	0.5
IM-13	plastic	A	0.9
IM-14	plastic	A	1.8
IM-15	plastic	B	0.7
IM-7	plastic	Control	0.6
IM-9	steel pipe	Control	0.5

Five slope inclinometer casings were installed by MoDOT drilling crews during the period July 7-9, 2002. All casings were installed to a depth of approximately 19-ft (5.8-m) below ground surface to extend below the toe of the slope and ensure adequate founding in the very stiff clay. One casing was installed within each section of the main slide area in close proximity to the instrumented reinforcing members. The fifth casing was installed in the control slide area approximately midway between the two instrumented members.

Figure 6.10 Photograph of US36-Stewartsville control slide during installation of instrumented reinforcing members.

Two clusters of standpipe piezometers were installed at the site during the same period. Both clusters were placed near the middle section of the slide between Sections B and C. Each cluster contained three piezometers placed within a single borehole. Piezometers P-1, P-2, and P-3 were placed in a cluster located in the upper third of the slope and screened at depths of 14-ft, 9-ft, and 4-ft (4.2-m, 2.7-m, and 1.2-m), respectively (Figure 5.24). Piezometers P-4, P-5, and P-6 were placed in a cluster located in the lower third of the slope and screened at similar depths.

Instrumentation to monitor the moisture conditions and soil suction within the slope was installed at 7 locations across the main slide area on August 23, 2002. As was done at the I435-Wornall Road site, a vertical array of ThetaProbes® and Equitensiometers® was installed at location M-4 near the center of the slide area (Figure 6.9) to establish an essentially continuous record of moisture/suction conditions within the slope. A Profile Probe® access tube was also installed at location M-4. Additional Profile Probe® access tubes were installed at the remaining locations shown in Figure 6.9 (designated M-1 through M-7) to provide data on the vertical and lateral distribution of moisture conditions across the slide area.

6.5. Field Performance

Instrumentation at US36-Stewartsville test site has been monitored at regular intervals for 15 months. The following sections describe the results obtained for the different types of instrumentation utilized at the site.

6.5.1. Precipitation at the US36-Stewartsville Site

Figure 6.11 shows the daily and monthly precipitation totals recorded at the Rosecrans Memorial Airport in St. Joseph Missouri, approximately 20 miles (32-km) west of the site. The general pattern of precipitation at the site has been normal with relatively wet springs and relatively dry winters. However, the overall magnitude of precipitation has been much lower than normal with only four months out of the past fifteen experiencing near normal precipitation. Precipitation in the remaining eleven months has been below normal resulting in a significant rainfall deficit in the area. It is also notable that few of the precipitation events experienced at the site since installation have been very heavy events with most events being less than 1-in (2.5-cm) of precipitation and the most severe event being approximate 2-in (5.0-cm) of precipitation.

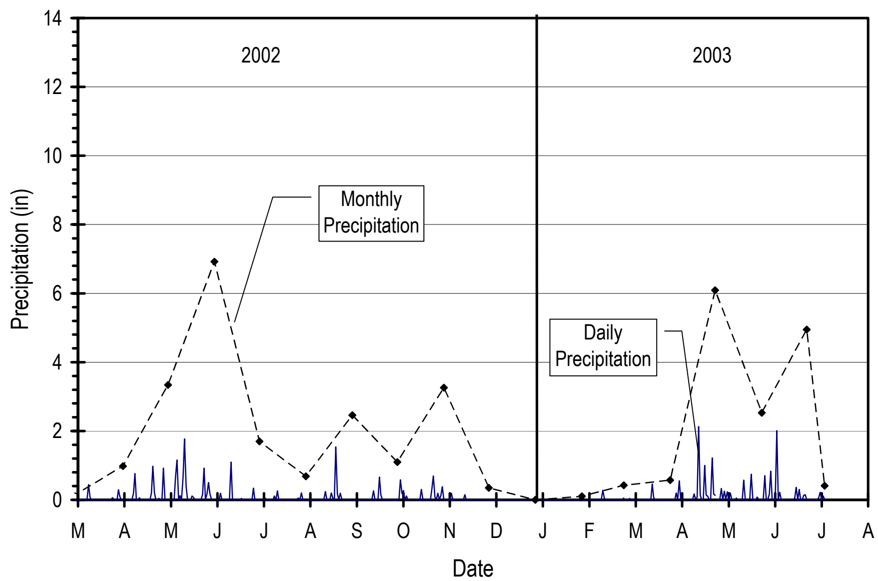


Figure 6.11 Monthly and daily precipitation from St. Joseph Rosecrans Memorial Airport located approximately 20 miles west of the US36-Stewartsville test site.

6.5.2. Piezometers and Moisture Sensors

Water levels measured in each of the six piezometers installed at the site are plotted in Figure 6.12. Following an initial period of equalization, piezometer P-1 has generally shown water levels between 11- and 13-ft (3.3- and 3.9-m) below ground surface. Piezometers P-2 and P-3 have been dry since installation. Piezometers P-4 and P-5, located near the toe of the slope, have consistently shown piezometric water levels approximately 4-ft (1.2-m) below the surface while piezometer P-6 has shown similar water levels or been dry (the base of piezometer P-6 is very near the water level indicated by P-4 and P-5). Overall, the piezometric water levels have not exhibited significant response to precipitation events. However, this is somewhat expected given the rainfall deficit that has been experienced in the area.

Measurements taken for the soil moisture and soil suction sensors located near the center of the slide area are plotted in Figure 6.13. In contrast to the experience with

Equitensiometers® at the I435-Wornall Road test site, the Equitensiometers® at the US36-Stewartsville site have produced excellent data. This is primarily attributed to the fact that the I435-Wornall Road slope has remained very wet and likely saturated since installation while the US36-Stewartsville slope has been unsaturated. The trends in volumetric water content and soil suction have been very similar throughout the period of monitoring, which provides some confidence that the sensors are providing accurate data. In the upper Equitensiometer®, pore water pressures were observed to decrease dramatically during the first few months of monitoring. The upper Equitensiometer® then showed a rapid increase in pore water pressures in late October to early November 2002 which appears to be a response to a week of moderate rainfall. Both the pore water pressures and soil moisture contents subsequently decreased over the winter months before increasing again in response to increased rainfall in April and May of 2003. The lower Equitensiometer® and ThetaProbe® have been somewhat less responsive to specific rainfall events (as would be expected) but they both do show decreasing pore pressures and volumetric water contents over the dry winter months followed by a relatively rapid increase in pore pressures and water content during Spring 2003.

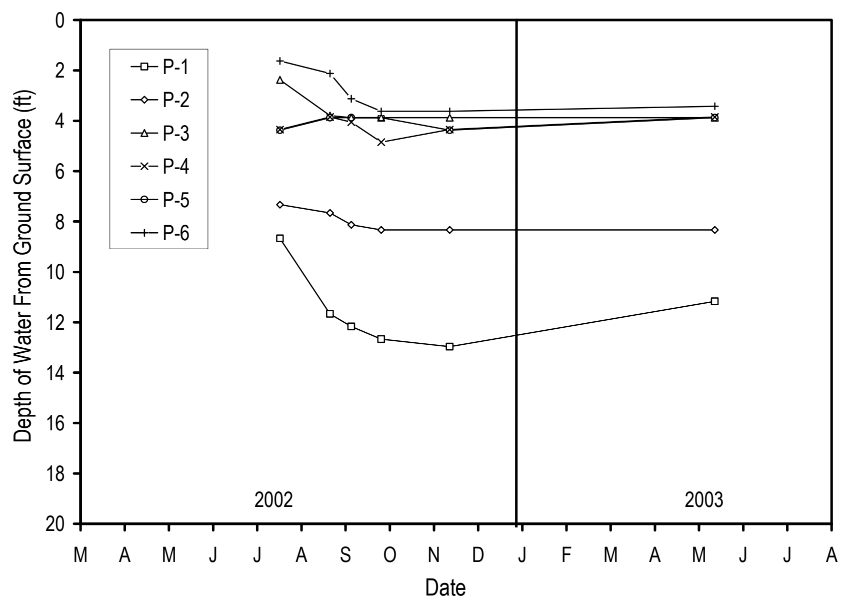


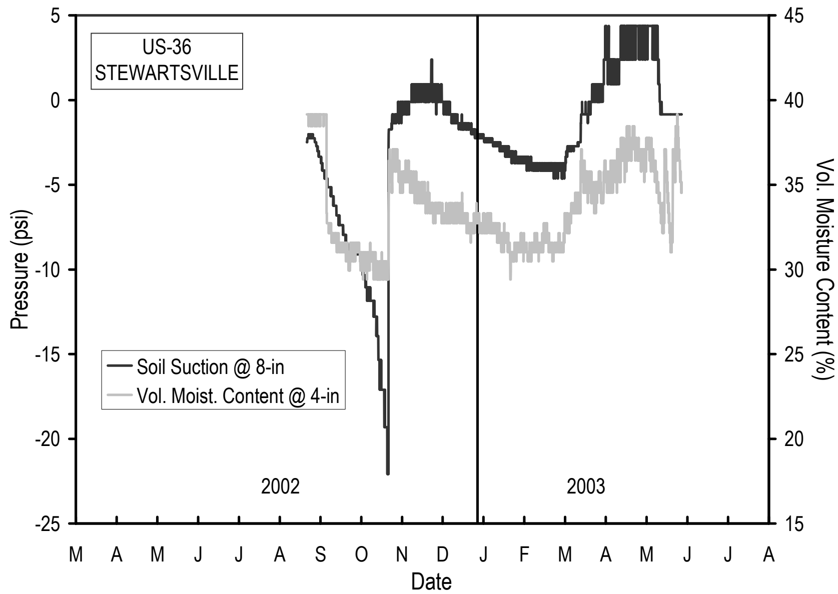
Figure 6.12 Piezometric water levels measured at US36-Stewartsville site.

6.5.3. Slope Inclinometers

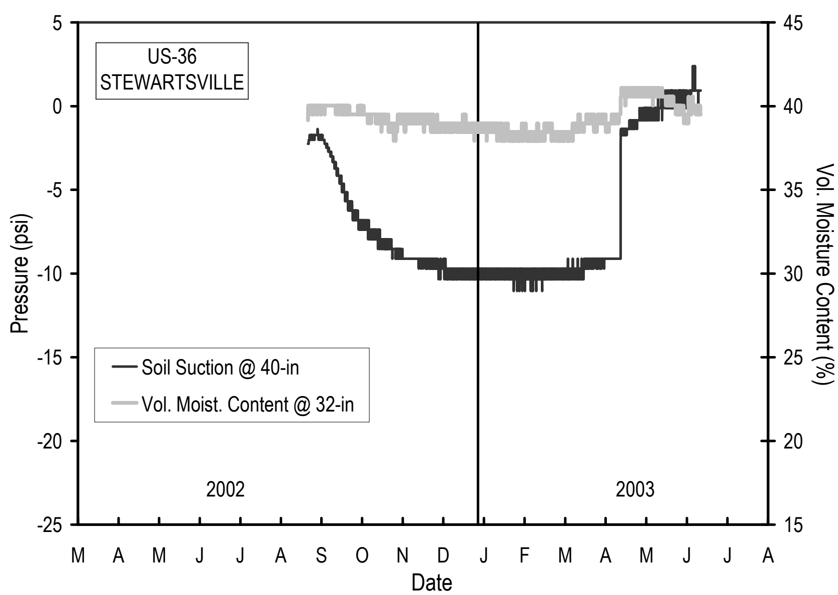
Lateral deformations determined from inclinometer I-2 at the US36-Stewartsville test site are plotted in Figure 6.14. Other inclinometers at the site had similar deformation profiles. In general, the deformation profiles show maximum deformation occurring near the ground surface with continuously decreasing deformations at greater depths.

Figure 6.15 shows the maximum lateral deflections measured for each of the inclinometers as a function of time. This figure indicates that deflections were generally small over the first 11 months following installation. Some inclinometers showed slight upslope movements during the first few months following installation, but these are likely due to "settling in" of the inclinometer casings rather than actual upslope movements.

Deflections increased slightly between the March and May 2003 readings, which appears to correlate with the heavier precipitation experienced during that time. However, additional readings are needed to confirm this correlation. It is also noteworthy that very little difference is observed in the magnitudes and trends of deformations for the four different test sections and the control slide throughout the period of monitoring. This suggests that resistance from the reinforcing members has yet to be mobilized to measurable levels.



(a) readings for shallow sensors



(b) readings for deeper sensors

Figure 6.13 Soil suction and volumetric water content measurements from the US36-Stewartsville test site: (a) shallow sensors and (b) deeper sensors.

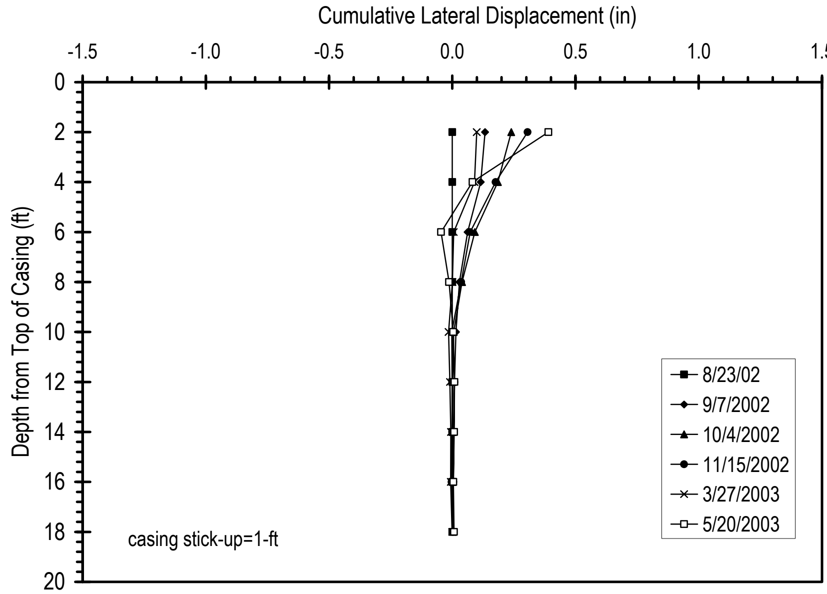


Figure 6.14 Lateral deflection profile for inclinometer I-3 at US36-Stewartsville site.

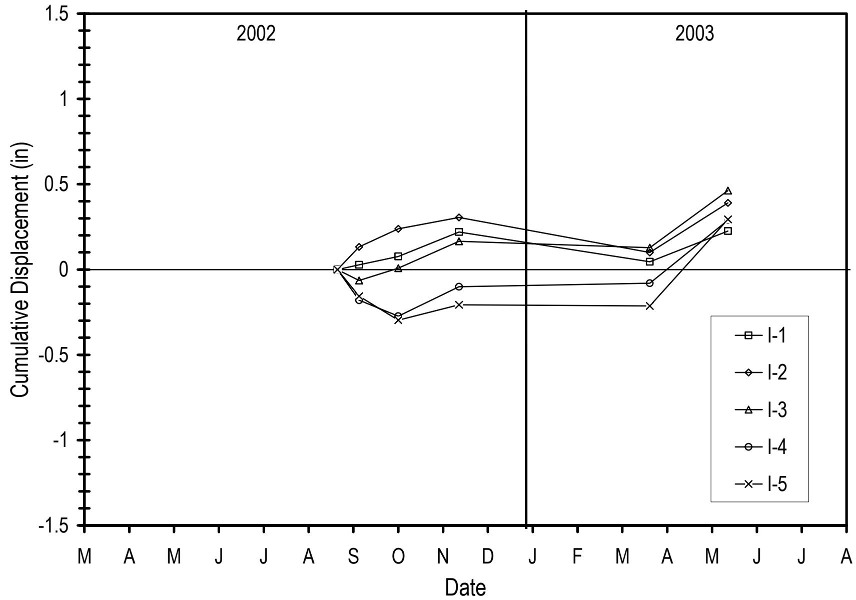
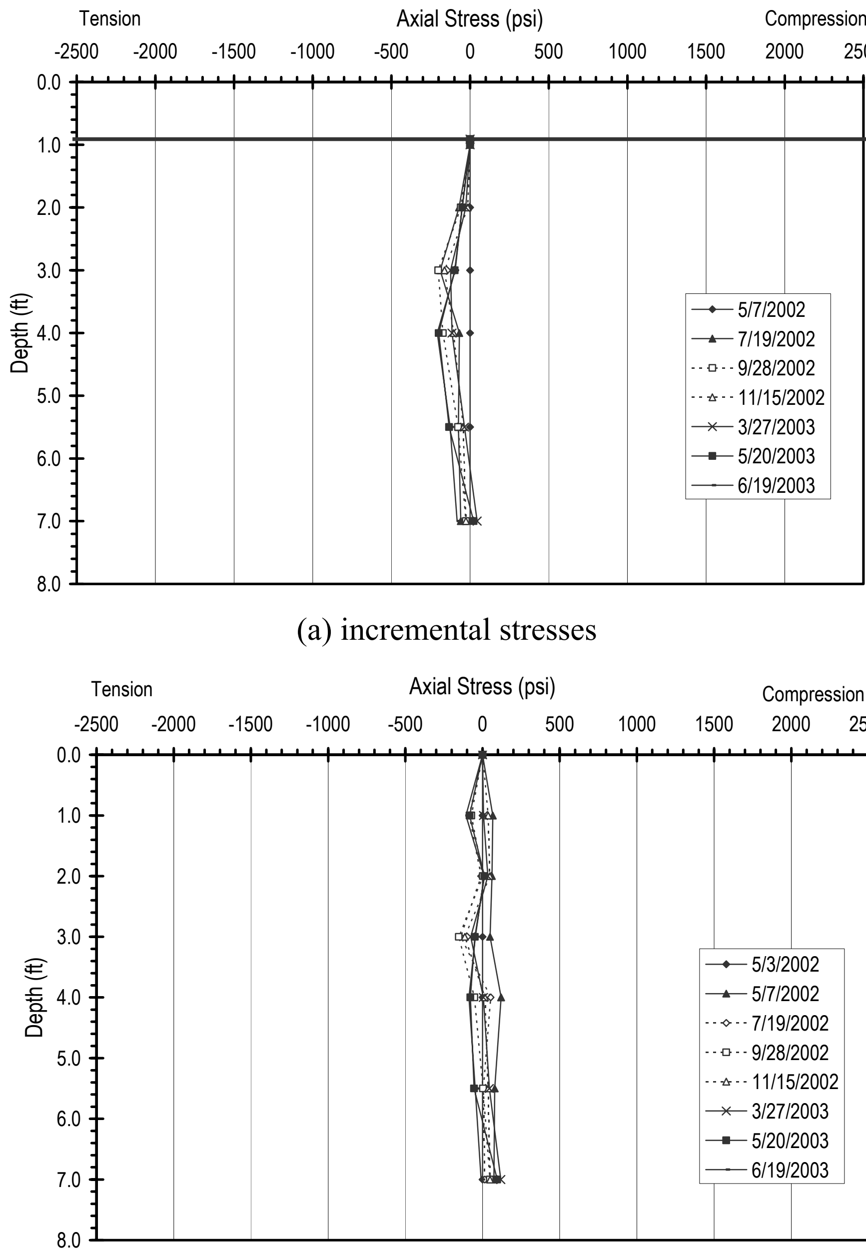


Figure 6.15 Cumulative lateral deflections for inclinometers I-1 through I-5 at depth of 1-ft for US36-Stewartsville site.

6.5.4. Instrumented Reinforcing Members

Figure 6.16 shows the distribution of incremental and overall axial stresses determined for instrumented member IM-13 in Section A of the slope. The distribution for this member is generally a parabolic distribution of stress with small axial stresses near the ends of the member and maximum axial stress near the center of the member. All other instrumented members except member IM-15, which experienced loss of a large number of strain gages shortly after installation, produced similar distributions of axial stresses.

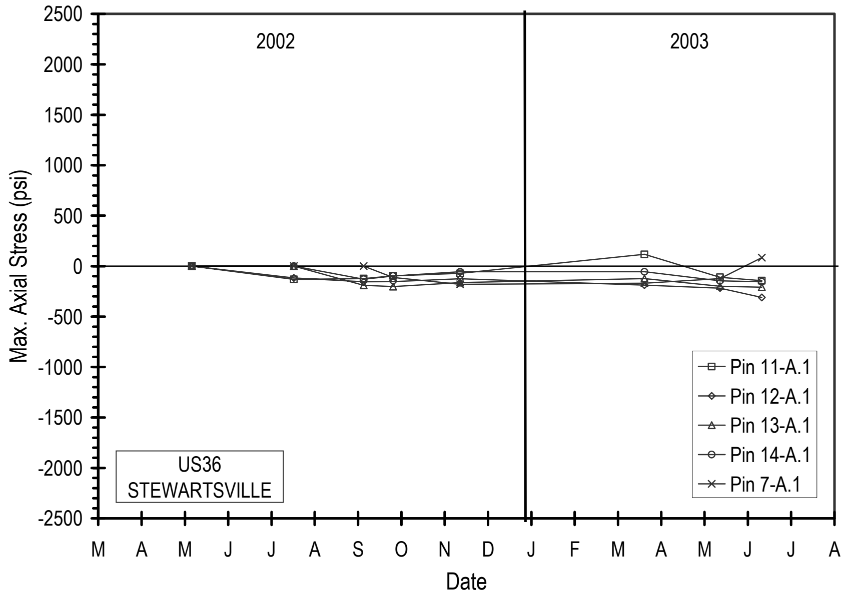


(a) incremental stresses

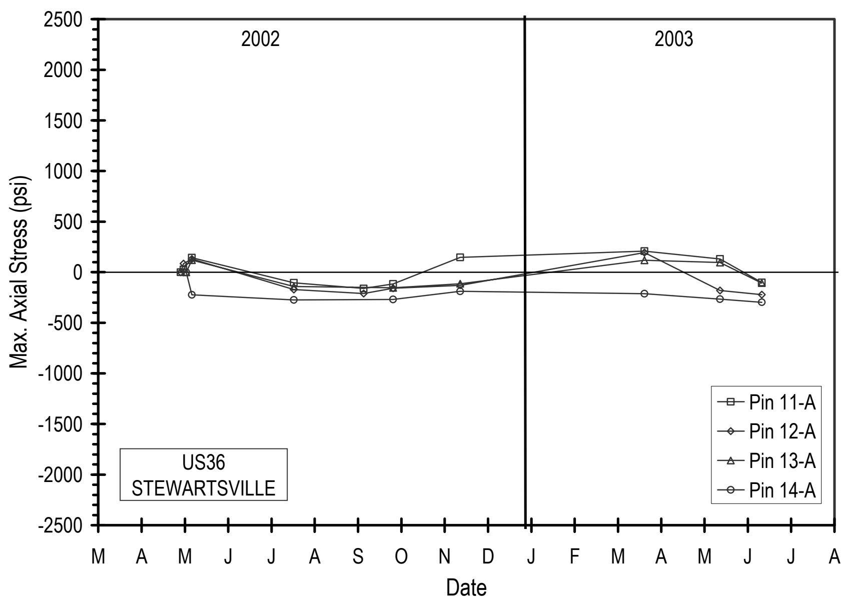
(b) overall stresses

Figure 6.16 Measured axial stresses in instrumented member IM-13 at the US36-Stewartsville test site: (a) incremental stresses and (b) overall stresses.

Figure 6.17 shows the maximum incremental and overall axial stresses in all instrumented members plotted versus time. As was the case with members at other sites, initial stresses imposed during installation were generally compressive, but all changes in stress (incremental stresses) have been largely tensile. However, it is notable that the magnitudes of the initial stresses imposed during installation (<300-psi) are generally small as are the magnitudes of the incremental stresses since installation. This is in contrast to measurements from the I435-Wornall Road site where the initial stresses were small, but the incremental changes in stress were much larger (-1000- to -2000-psi at I435 as compared with less than -300-psi at US36). The reasons for this discrepancy are not known at this time.



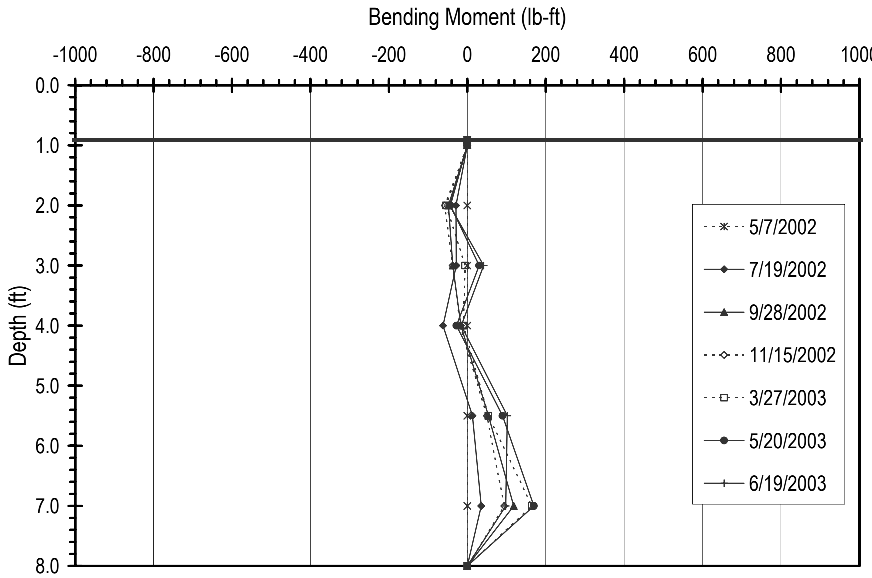
(a) incremental stresses



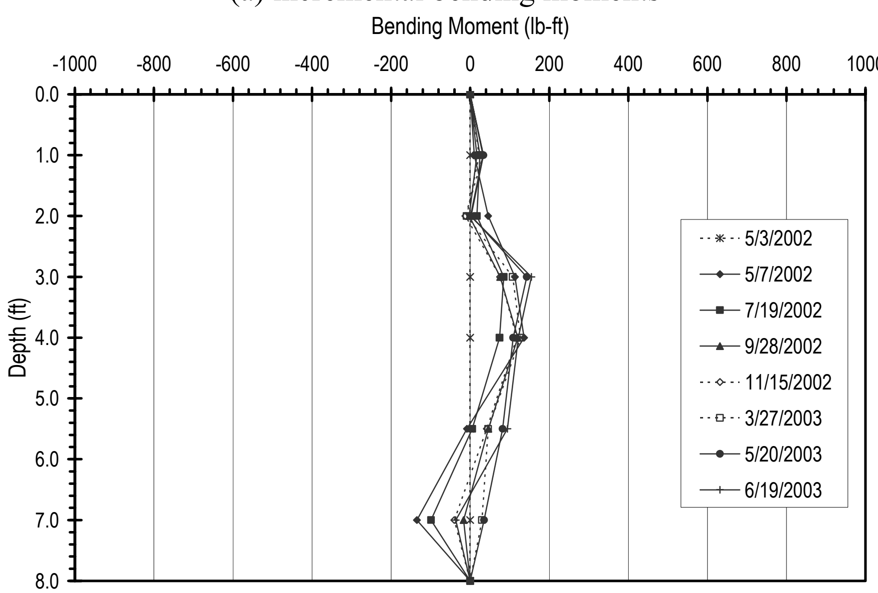
(b) overall stresses

Figure 6.17 Maximum axial stress in instrumented members at US36-Stewartsville test: (a) incremental stresses and (b) overall stresses.

Measured incremental and overall bending moments for instrumented member IM-13 are plotted in Figure 6.18. The incremental bending moments for the member take on a characteristic S-shape with negative bending moments near the top of the member and positive bending moments near the tip. In contrast, overall bending moments are generally positive along the entire length of the member except near the tip where early readings showed slightly negative bending moments. This shows that measurable bending moments are induced in the member during installation. While such moments contribute to the overall moments in the member, they are not representative of moments due to slope movements and must be considered separately.



(a) incremental bending moments



(b) overall bending moments.

Figure 6.18 Measured bending moments in instrumented member IM-13 at the US36-Stewartsville test site: (a) incremental bending moments and (b) overall bending moments.

Incremental bending moments measured for instrumented members IM-11, IM-14, and IM-7 had distributions with the characteristic S-shape similar to that shown in Figure 6.18 for member IM-13. Member IM-12 had a somewhat different distribution of incremental bending moments as shown in Figure 6.19. For this member, the distribution of bending moments has a parabolic shape with all negative bending moments and with a maximum bending moment occurring in the lower portion of the member. Overall bending moments for the different instrumented members varied somewhat randomly, which is expected given that different members installed in different locations may have different bending moments induced by the installation.

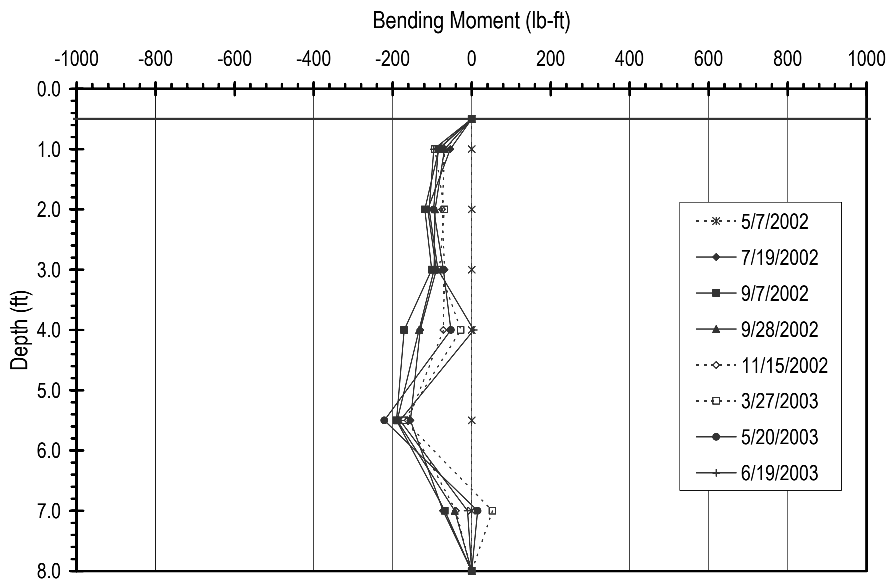
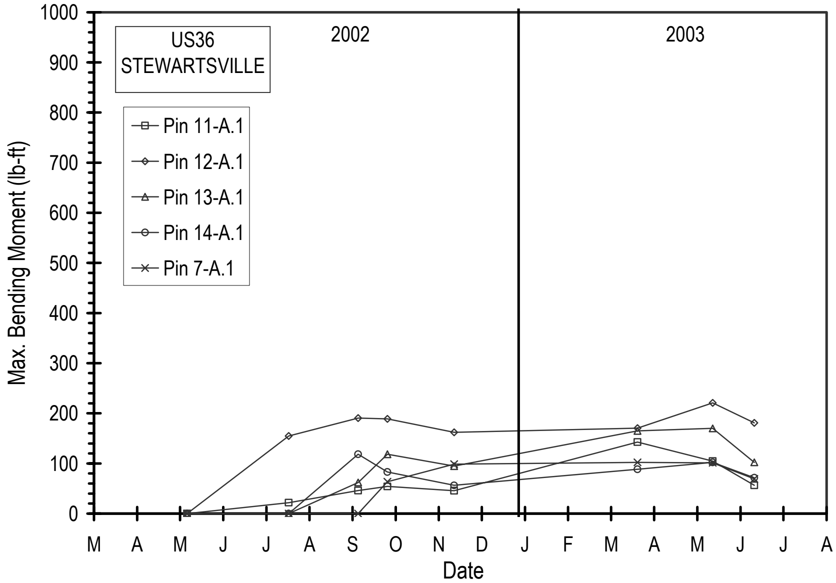


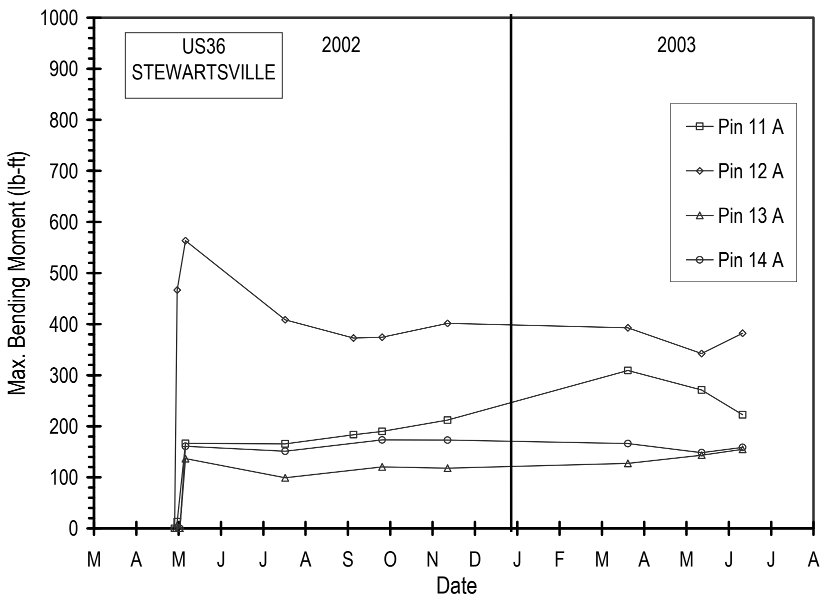
Figure 6.19 Measured incremental bending moments in instrumented member IM-12 at the US36-Stewartsville test site.

The maximum incremental and overall bending moments for each instrumented member at the US36-Stewartsville site are plotted as a function of time in Figure 6.20. Overall bending moments are plotted only for those members where initial readings were taken prior to installation. As shown in Figure 6.20a, the incremental bending moments for all members except member IM-12 were small for the first few months following installation. Incremental bending moments for these members have increased slightly since that time but still remain small and well below the moment capacity of the reinforcing members. Incremental bending moments for member IM-12 appear to increase just after installation, after which the incremental moments have been essentially constant.

Overall bending moments at the US36-Stewartsville site have different trends as shown in Figure 6.20b. Initial bending moments induced during installation for members IM-11, IM-13, and IM-14 were all around 150-lb-ft (200-N-m) while initial bending moments for member IM-12 were approximately 500-lb-ft (680-N-m). Since installation, overall bending moments for members IM-13 and IM-14 have remained essentially constant while overall bending moments for member IM-11 increased and overall bending moments for member IM-12 decreased. The reason for the overall bending moments decreasing with time is that the initial moments induced during installation were positive bending moments while the changes in moments (incremental bending moments) were negative (Figure 6.19), which produced a net reduction in the overall bending moments because of the different signs. For all members, the incremental bending moments induced since installation have generally been small (<200-lb-ft) which is consistent with the fact that lateral deformations have been small as a result of dry soil conditions since installation.



(a) incremental bending moments



(b) overall bending moments

Figure 6.20 Maximum bending moment in instrumented members at US36-Stewartsville test site: (a) incremental bending moments and (b) overall bending moments.

6.6. Status of Instrumentation at US36-Stewartsville Site

Overall the instrumentation installed at the US36-Stewartsville site has performed well since installation. The slope inclinometers and standpipe piezometers continue to be in good condition and are expected to continue to provide valuable information, particularly when rainfall levels at the site increase. The moisture sensors also appear to be performing well and have provided valuable information on changes in moisture conditions within the slope throughout monitoring. As was the case at the I435-Kansas City sites, the Profile Probe® data has had significant scatter, which has resulted in some difficulty in interpreting the data. Efforts to establish procedures to make better use of this data are ongoing. Aside from instrumented member IM-15, all of the instrumented members at the US36-Stewartsville site have performed well. Relatively few of the strain gages have become inoperable, and we expect to continue acquiring data from the instrumented members for the foreseeable future.

6.7. Summary

In this chapter, the activities undertaken to construct, instrument, and monitor the performance of the US36-Stewartsville test site have been described. Unlike previously established sites, the US36-Stewartsville site was stabilized using a variety of different reinforcement patterns to permit direct comparison of the effectiveness of alternative measures. To date, all of the different stabilization schemes continue to be performing well. Precipitation at the site since installation has been well below normal, which has resulted in relatively small movements and relatively small loads in the reinforcing members as would be expected. Additional monitoring is ongoing so that the performance of the different stabilized sections can be reliably compared when precipitation increases to normal, or above normal, levels.

Chapter 7. I70-Emma Site

The fourth slope stabilized during Phase II of the project is located at the I70-Emma site. Two slide areas at this site were stabilized during Phase I of the project. Two additional slide areas were simply regraded to the original slope geometry to serve as control sections for the stabilized areas. Both control sections subsequently experienced failures. One of these control areas was selected for stabilization in Phase II to evaluate the potential for using more widely spaced reinforcement configurations to stabilize surficial slides. In this chapter, the activities undertaken to establish the three test areas at the I70-Emma site are described with particular focus on activities undertaken during Phase II of the project. Activities undertaken during Phase I are described in a previous report (Loehr et al, 2001) and are therefore presented only to the extent necessary for completeness.

7.1. Site Characteristics

The I70-Emma site is located on Interstate 70 approximately 65 miles (105-km) west of Columbia Missouri and approximately 1 mile north of the city of Emma Missouri. Figure 7.1 shows an air photo of the area indicating the location of the site. The slope is an embankment that forms the eastbound entrance ramp to Interstate 70. The embankment is approximately 22-ft (6.7-m) high with side slopes varying from 2.5H:1V to 2.2H:1V and is composed of mixed lean and fat clays with scattered gravel, cobbles, and construction rubble (concrete and asphalt). Prior to being selected for stabilization as part of this project, the embankment had experienced recurring slides in four areas of the embankment over the past decade or more. Figure 7.2 shows a plan view of the site indicating locations of the four slide areas denoted S1, S2, S3, and S4. Figure 7.3 shows a photograph of the south side of the embankment following the failures that occurred prior to stabilization during Phase I. Previous stabilization attempts consisting of regrading the slope, dumping concrete rubble over the crest of the embankment, and replacing soils near the toe with construction rubble were unsuccessful.

Figure 7.1 Air photo of Interstate 70 near Emma Missouri taken March 8, 1997 showing location of I70-Emma site (from USGS).

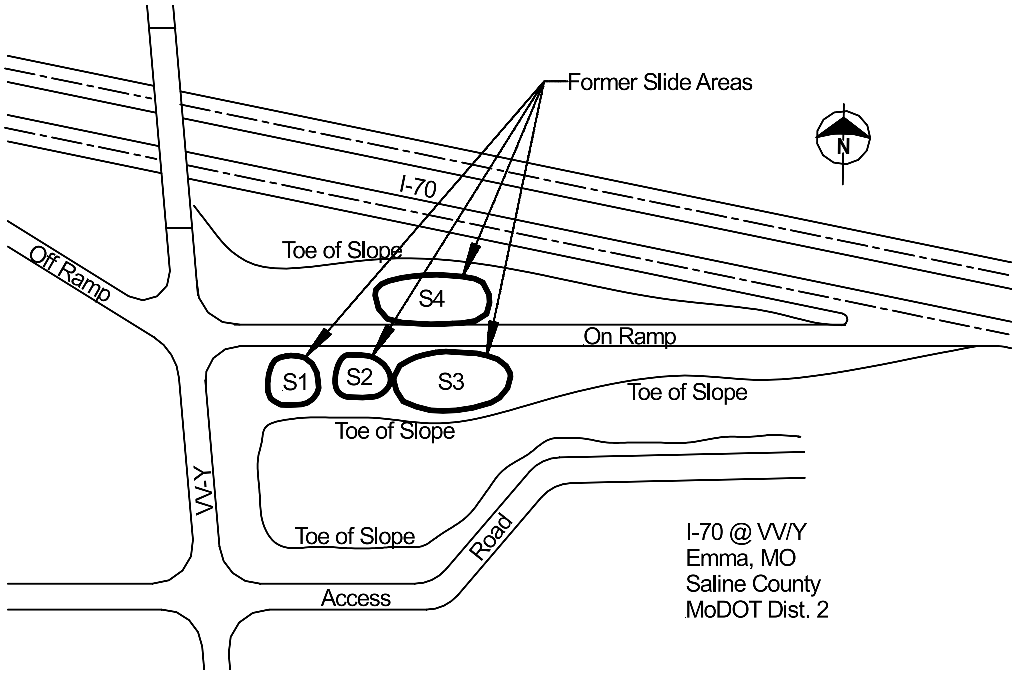


Figure 7.2 Plan view of I70-Emma site showing slide areas S1, S2, S3, and S4.

Figure 7.3 Photograph of south side of embankment at I70-Emma site showing slide areas S1 (left), S2 (center), and S3 (right).

Boring and sampling activities at the I70-Emma site were performed on June 1-3, 1999 prior to stabilization during Phase I. A total of 11 borings were made across the site to depths ranging from 10- to 33-ft (3- to 10-m). A plan view of the site showing boring locations and boring logs are provided in Appendix C. In each boring, continuous 3-in (7.6-cm) diameter Shelby tube samples were taken, extruded in the field for identification and field testing, and then wrapped in aluminum foil and waxed for transport to the University of Missouri Geotechnical Engineering Laboratories for subsequent laboratory testing. The borings indicate the slope is composed of a mixture of lean to fat clays of variable consistency with scattered fine gravel and cobbles that are presumed to be construction rubble remaining from previous stabilization attempts. Several of the borings made near slide areas S1 and S2 indicated groundwater was present below the elevation of the toe of the slope; all other borings were dry throughout the site investigation. Standard Penetration tests performed near the base of the embankment produced SPT N_{60}-values between 3 and 8. Two other SPT tests performed below the base of the embankment produced N_{60} in excess of 40.

Moisture contents determined from samples taken in the field indicate that field moisture contents were essentially constant with depth throughout the embankment. Measured moisture contents ranged from 14 to 34 percent but the vast majority of values were between 20 and 25 percent. Atterberg limits determined for samples from the site indicate the soils have liquid limits (LL) from 39 to 60, plastic limits (PL) from 19 to 27, and plasticity indices (PI) from 10 to 41. The soils generally classified as either CL or CH in the USCS, although one sample classified as ML. No clear trends were observed in the Atterberg limits for soils from the site which indicates the embankment is composed of an essentially random mixture of soils.

Mohr-Coulomb effective stress shear strength parameters for the Emma site soils were determined from both triaxial compression and direct shear tests. All but two of these tests were performed during Phase II of the project. Figure 7.4 shows the stress paths determined from consolidated-undrained ($\overline{\text{CU}}, \overline{R}$) and consolidated-drained (CD,S) type triaxial compression tests along with upper bound and lower bound failure envelopes established from the test results for the surficial soils and soils at greater depths. A summary of the drained effective stress strength parameters for the surficial and deeper soils is given in Table 7.1. These tests indicate that $\bar{c}$ for the surficial soils is equal to approximately 100-psf (4.8-kPa) and $\bar{\phi}$ is equal to 23 degrees while for the deeper soils $\bar{c}$ ranges from 170- to 365-psf (8.1- to 17.5-kPa) and $\bar{\phi}$ is approximately 25 degrees.

Table 7.1 Summary of Mohr-Coulomb effective stress strength parameters from direct shear and triaxial compression tests on specimens from the I70-Emma test site.

			upper bound		lower bound		direct shear	
Stratum	Depths	Sample Numbers	$\bar{c}$ (psf)	$\bar{\phi}$ (°)	$\bar{c}$ (psf)	$\bar{\phi}$ (°)	$\bar{c}$ (psf)	$\bar{\phi}$ (°)
Surficial clay	< 4.0-ft	274 313	96	23	--	--	202	14
Deeper clay	> 4.0-ft	277, 278 286, 287 284, 289	364	25	170	25	101	14

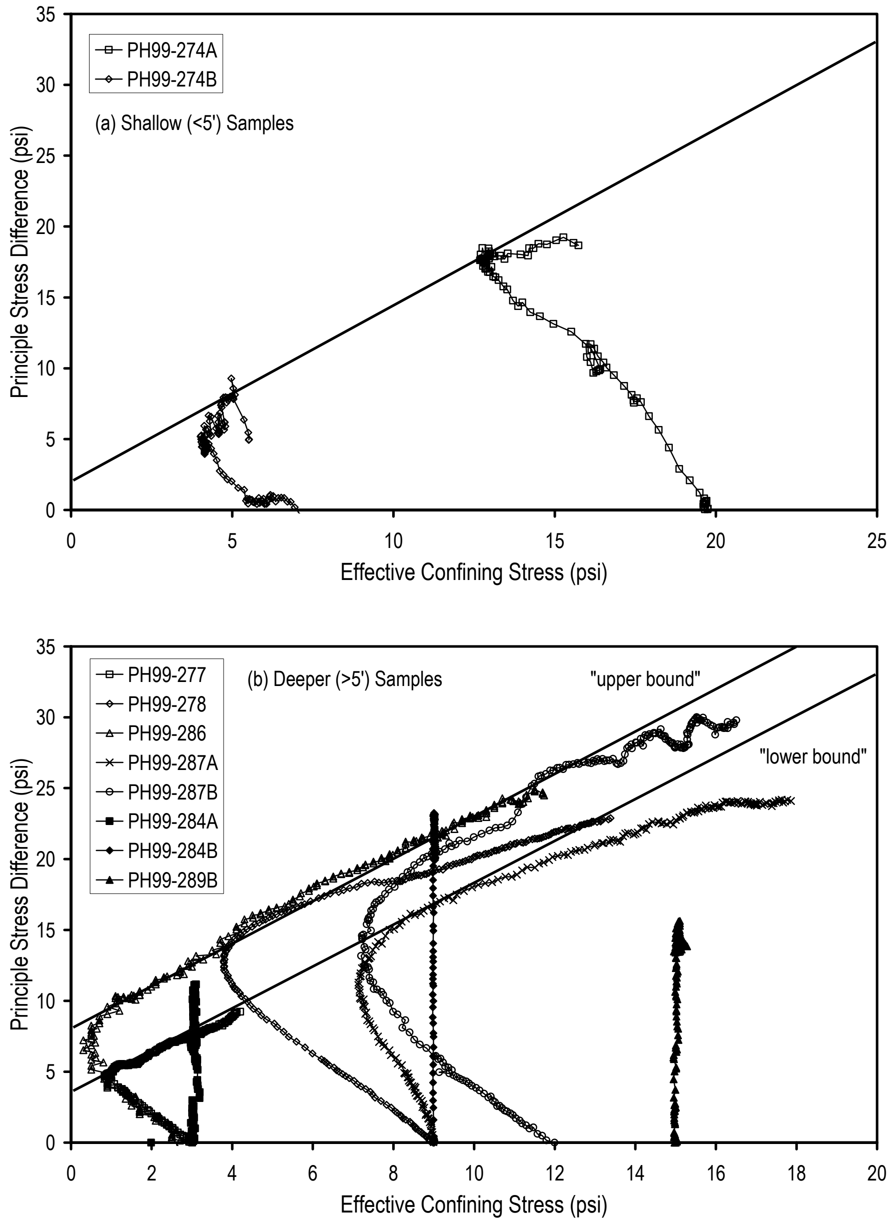


Figure 7.4 Summary of triaxial test results for specimens from the I70-Emma site: (a) shallow samples and (b) deeper samples.

Figure 7.5 shows the results of the drained direct shear tests on two samples from the I70-Emma site with peak shear strength failure envelopes determined for each sample. Mohr-Coulomb effective stress strength parameters for these envelopes are shown in Table 7.1. These values indicate that the both samples had $\bar{\phi}$ of 14 degrees, a value that is significantly lower than $\bar{\phi}$ obtained from the triaxial test results. Values of $\bar{c}$ ranged from 100- to 200-psf (4.8- to 9.6-kPa).

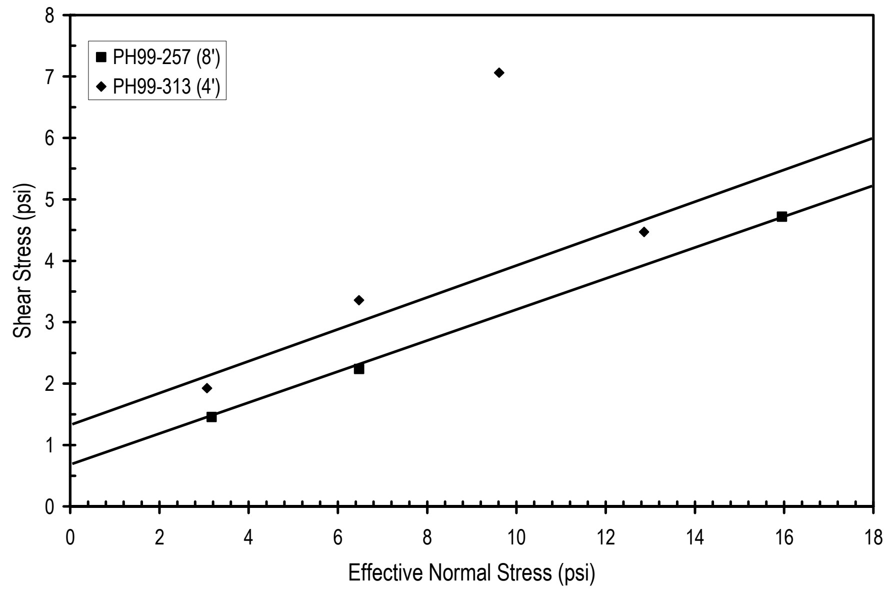


Figure 7.5 Summary of direct shear test results for specimens from the I70-Emma site.

7.2. Selection of Stabilization Schemes

The stabilization schemes utilized at the I70-Emma site in both Phase I and Phase II were established based on a limited number of stability analyses. The following sections described the stability analyses performed and the stabilization schemes selected for slide areas S1 and S2 during Phase I and slide area S3 during Phase II, respectively.

7.2.1. Stabilization Schemes for Slide Areas S1 and S2

The schemes utilized to stabilize slide areas S1 and S2 during Phase I of the project were determined from preliminary analyses performed using back-calculated strength parameters. In these analyses, the slope was assumed to be essentially homogenous and the soil was assumed to have negligible cohesion intercept (i.e. $\bar{c} = 0$) under fully drained conditions. Pore water pressures within the slope were assumed to be negligible. Based on these assumptions, the back-calculated value of $\bar{\phi}$ was determined to be approximately 22 degrees. These conditions were then used to evaluate factors of safety for various reinforcement configurations generally composed of members placed in a uniform grid across the entire slope. Factors of safety determined from these analyses ranged from 1.05 for a 6-ft longitudinal by 6-ft transverse (1.8-m by 1.8-m) grid of reinforcement to 1.43 for reinforcement placed on a 1-ft by 1-ft (0.3-m by 0.3-m) grid (Liew, 2000).

The reinforcement configurations selected for stabilization of slide areas S1 and S2 are shown in Figure 7.6. Both selected schemes included members placed on a 3-ft by 3-ft (0.9-m by 0.9-m) staggered grid. However, members for slide area S1 were inclined perpendicular to the face of the slope, while members for slide S2 were inclined vertically. The factor of safety for both of these reinforcement schemes was estimated to be approximately 1.2 based on calculations performed using the back-calculated soil conditions.

Calculations performed subsequent to the installation considering the potential for a perched water condition produced a similar factor of safety.

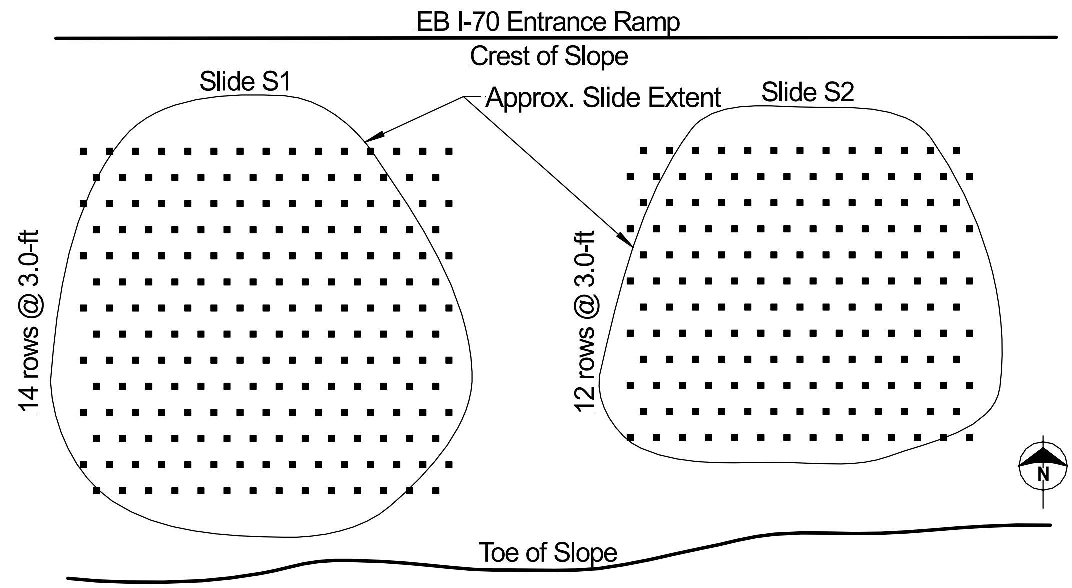


Figure 7.6 Plan view of slide areas S1 and S2 at the I70-Emma site showing selected layout of reinforcing members.

7.2.2. Stabilization Scheme for Slide Area S3

Since the stabilization schemes used for slide areas S1 and S2 during Phase I had proven to be effective for over two years, the stabilization scheme for slide area S3 was selected to evaluate the potential for stabilizing the slide area using more widely spaced reinforcement configurations. Figure 7.7 shows the final selected configurations. The slide area was separated into four sections, denoted Sections A through D, with different reinforcement schemes utilized in each section. In Section A, members were placed on a 4.5-ft by 3.0-ft (1.4-m by 0.9-m) longitudinal by transverse staggered grid. A 4.5-ft by 6.0-ft (1.4-m by 1.8-m) grid was used in Section B, a 6.0-ft by 6.0-ft (1.8-m by 1.8-m) grid in Section C, and a 6.0-ft by 4.5-ft (1.8-m by 1.4-m) grid in Section D.

Factors of safety for each of the reinforcement schemes were calculated for two different possible sets of slope conditions as summarized in Table 7.2. The first set of conditions, referred to as stability condition A, was those determined from back-analyses described above. The second set of conditions, stability condition B, considered the two layer profile shown in Figure 7.8 with a perched water condition within the upper layer. For these analyses, the upper layer was assumed to have $\bar{c}$=95-psf (4.5-kPa) and $\bar{\phi}$=15 degrees while for the lower layer had $\bar{c}$=310-psf (14.8-kPa) and $\bar{\phi}$=22 degrees and the piezometric line for the upper layer was assumed to be at the ground surface. The factor of safety for these conditions without reinforcement is 1.0. As shown in Table 7.2, factors of safety calculated for Section A range from 1.10 to 1.16, Section B from 1.03 to 1.10, Section C from 1.01 to 1.06, and Section D from 1.02 to 1.08.

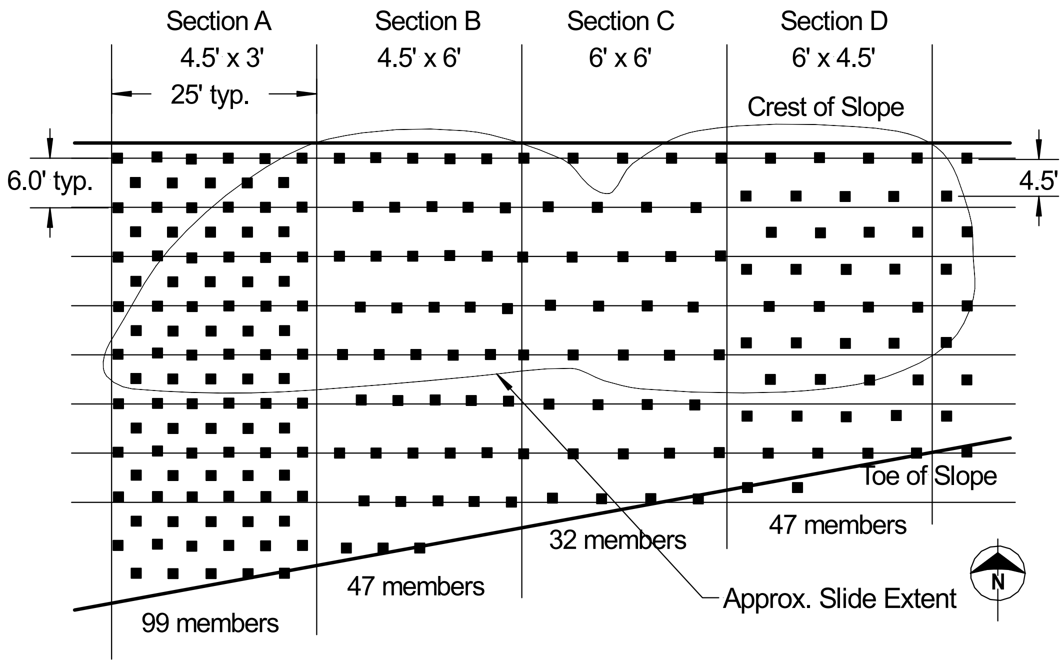


Figure 7.7 Plan view of selected stabilization schemes for slide area S3 at the I70-Emma test site.

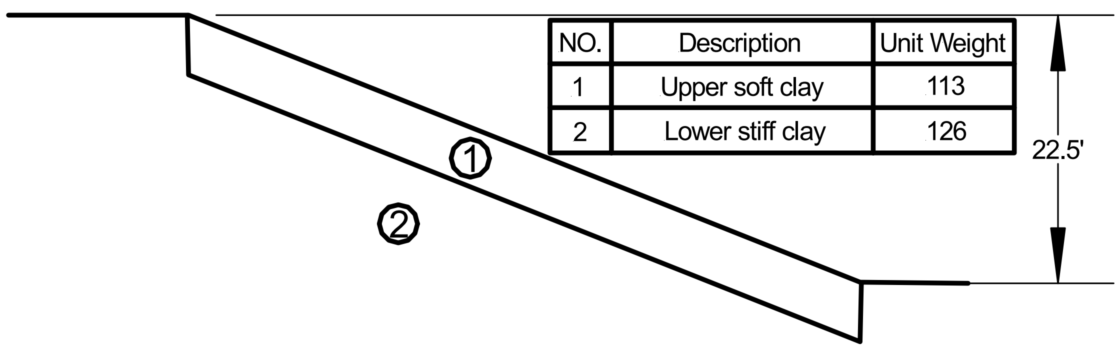


Figure 7.8 Cross-section assumed for stability condition B at the I70-Emma test site.

Table 7.2 Summary of estimated factors of safety for Sections A through D in slide area S3 and slide areas S1 and S2 at the I70-Emma test site

Rein. Spacing (ft)	Slope Section	Factor of Safety for Respective Stability Condition A	B
3.0L x 3.0T [1]	S1, S2	1.20	1.21
4.5L x 3.0T	A	1.16	1.10
4.5L x 6.0T	B	1.10	1.03
6.0L x 4.5T	D	1.08	1.02
6.0L x 6.0T	C	1.06	1.01

[1] L and T denote spacing in longitudinal (strike) and transverse (dip) directions, respectively

7.3. Field Installation

Field installation activities at the I70-Emma site were performed at two different times. Slide areas S1 and S2 were stabilized in October and November 1999 during Phase I of the project. Slide area S3 was stabilized in January 2003 during Phase II. Field installation activities during these two periods are described in more detail in the following sections.

7.3.1. Installation in Slide Areas S1 and S2

Initial attempts to install reinforcing members at the I70-Emma site occurred on October 18, 1999. The equipment utilized at this time was a Case 580 backhoe with an Okada OKB 305 hydraulic hammer shown in Figure 7.9. While the backhoe mounted hammer was able to drive the recycled plastic members into the slope, it was extremely difficult to get the members installed without damaging them. Installation of 45 members was attempted using the backhoe, but 22 of these were broken during installation. The primary reason for the high incidence of member failure was that the equipment had no means for maintaining the alignment of the hammer and the reinforcing member other than the skill of the operator. As the member was installed, the backhoe boom follows an arc which requires that the hammer be continuously realigned to maintain alignment with the reinforcing member and prevent failure of the members in bending due to the misalignment. Doing so proved exceptionally difficult, particularly since it was difficult to maintain the equipment in a fixed position on the slope. An additional problem with this equipment included having substantial difficulty maneuvering into position on the slope which caused severe rutting and damage to the slope and previously installed reinforcing members. As a result of these problems, use of the backhoe mounted hammer was discontinued.

Figure 7.9 Backhoe mounted hammer used for initial installation attempts at the I70-Emma test site during Phase I.

Installation at the I70-Emma site was resumed in November 1999 using a Davey-Kent DK100B track-mounted hydraulic rock drill shown in Figure 5.11. This rig proved to be much more effective than the backhoe because the rig has a mast that maintains the alignment of the percussion hammer with the reinforcing member. The rig also proved to be much more maneuverable and caused less damage to the slope face, although the rig did have to be tethered to a truck located at the top of the slope when installing members on the steepest areas of the slope. A second rig, the Ingersoll Rand (IR) CM150 pneumatic rock drill (also shown in Figure 5.11) used at the I435-Kansas City and US36-Stewartsville sites, was also used at the I70-Emma site during this time. However, penetration rates achieved with the IR rig were significantly lower than those achieved with the Davey-Kent rig so its use was limited to installation of only a handful of members.

Using the Davey-Kent rig, a total of 154 members[1] were installed in slide area S1 and 163 in slide area S2 during the period November 11-22, 1999. Members in slide area S1 were installed approximately perpendicular to the slope face while members in slide area S2 were installed vertically. Most members were driven to full depth. However, near the toe of the slope where the soil had been previously replaced with concrete rubble and/or large aggregate, members generally encountered refusal at depths from 3- to 6-ft (0.9- to 1.8-m). The portions of members remaining above ground were subsequently cut off using a gas-powered chain saw. Figure 7.10 shows a photograph of the site near the end of installation.

Figure 7.10 Photograph of I70-Emma test set near the end of installation activities for slide areas S1 and S2.

While several mechanical problems delayed completion for several days, the overall performance of the track-mounted rig proved acceptable and all 317 members installed at the site were driven in a little over four working days. Figure 7.11 shows a frequency

[1] Most of the members planned for installation in the lowest three rows of reinforcement could not be installed because of underground obstacles.

distribution of the measured average penetration rates for members installed in slide areas S1 and S2. As shown in the figure, average penetration rates varied from 0.4- to 10.2-ft/min (0.1- to 3.1-m/min) with a mean of 4.4-ft/min (1.3-m/min) when considering data from both slide areas combined. Installation rates including set up time between member installations were relatively low given that this was the first test site, but a peak installation rate of approximately 80 members/day was achieved near the end of installation.

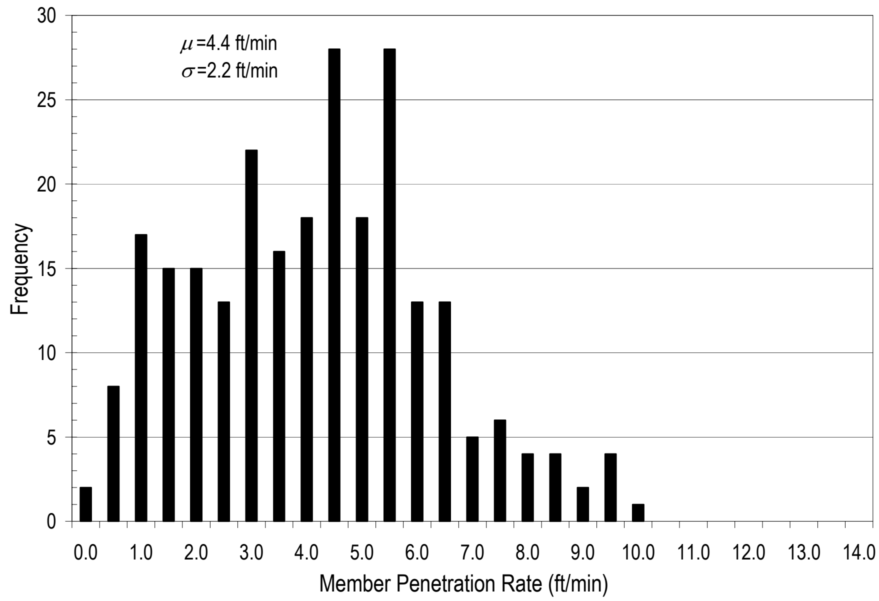


Figure 7.11 Frequency distribution of average penetration rates for recycled plastic members from Batch A2 installed in slide areas S1 and S2 at the I70-Emma test site. (μ=mean, σ=std. dev.)

7.3.2. Installation in Slide Area S3

Slide area S3 was regraded to the original slope configuration in early Fall 2002. Field installation in slide area S3 at the I70-Emma site took place on January 6-7, 2003. Reinforcing members were installed using two different pieces of equipment shown in Figure 7.12. The first piece of equipment was an Ingersoll Rand (IR) ECM350 track-mounted drill rig. This rig is a pneumatic hammer drill rig similar to the IR CM150 used at all previously completed sites. However, the ECM350 rig operates with higher air pressures and the drill mast is attached to an extendable boom that enables it to cover a larger area of the slope without requiring movement of the chassis. The extendable boom also permitted the equipment to move up the slope "face first" without tipping and eliminated the need to tether the equipment to the guard-rail or other support. The second piece of equipment utilized was a simple drop-weight device, the Daken Farm King, commonly used for driving fence or guard-rail posts mounted on a skid-steer loader. Both types of equipment performed exceptionally well which allowed installation to proceed more rapidly than had been possible at previous field sites.

A total of 199 reinforcing members were installed in slide area S3, 196 of which were recycled plastic members from Batch A10. Three members were 3-in diameter pressure-treated landscaping timbers installed to evaluate the "drivability" of these members. As was the case in slide areas S1 and S2, members installed near the toe of the slope generally met refusal at depths ranging from 3- to 6-ft (0.9- to 1.8-m) while members driven further up on the slope were driven to full depth. Recycled plastic members were generally driven without any significant problems and the overall installation was completed in less than two working days. Figure 7.13 shows a photograph of the site following the completed installation. Some slight "brooming" of the landscaping timbers was observed following installation of the timber members. However, the brooming is not expected to be significant in terms of the performance of the members.

Figure 7.12 Ingersoll Rand ECM350 pneumatic hammer drill (background) and drop-weight hammer rig (foreground) used to install reinforcing members in slide area S3 at the I70-Emma site.

Frequency distributions of the average penetration rates observed for both types of equipment used at slide area S3 are shown in Figure 7.14. Overall, the average penetration rates ranged from under 2-ft/min (0.6-m/min) to over 18-ft/min (5.5-m/min) with a mean of 6.5-ft/min (2.0-m/min) and standard deviation of 4.6-ft/min (1.4-m/min). No significance differences were observed in the installation rates for the two different types of equipment used. Daily installation rates for each rig exceeded 100-members/day indicating that both pieces of equipment were more effective than previously utilized installation equipment.

7.4. Instrumentation

Several types of instrumentation were installed at the I70-Emma test site during both Phases I and II. The following sections describe the instrumentation installed during each phase of the project.

Figure 7.13 Photograph of slide area S3 at the I70-Emma test site following completion of installation activities.

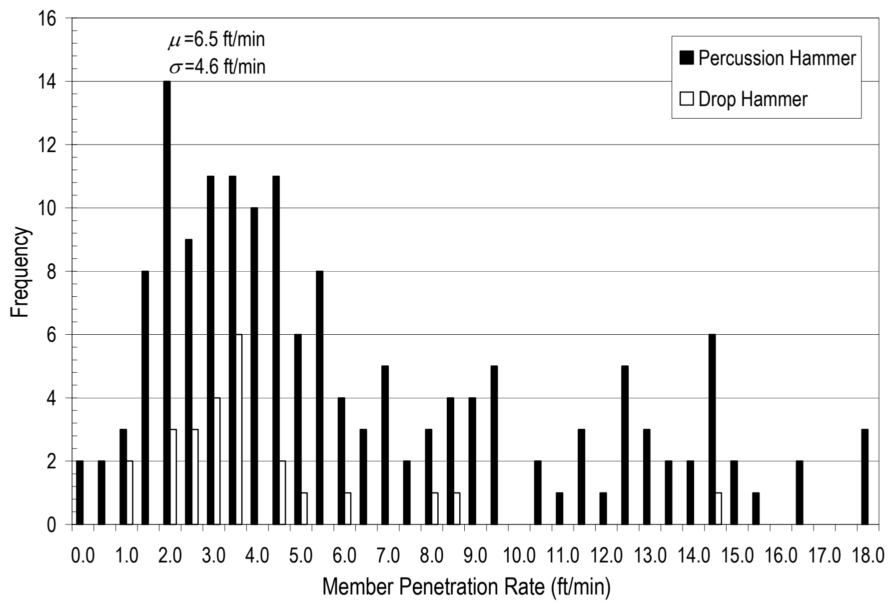


Figure 7.14 Frequency distribution of average penetration rates for recycled plastic members from Batch A2 installed in slide area S3 at the I70-Emma test site. (μ=mean, σ=std. dev.)

7.4.1. Instrumentation Installed During Phase I

Instrumentation installed at the I70-Emma test site during Phase I included instrumented reinforcing members to monitor loads in the reinforcing members, slope inclinometers to monitor lateral deformations in the slope, standpipe piezometers to monitor possible positive pore water pressures, and "jet-filled" tensiometers to monitor possible soil suction. Figure 7.15 shows the locations of the various types of instrumentation utilized.

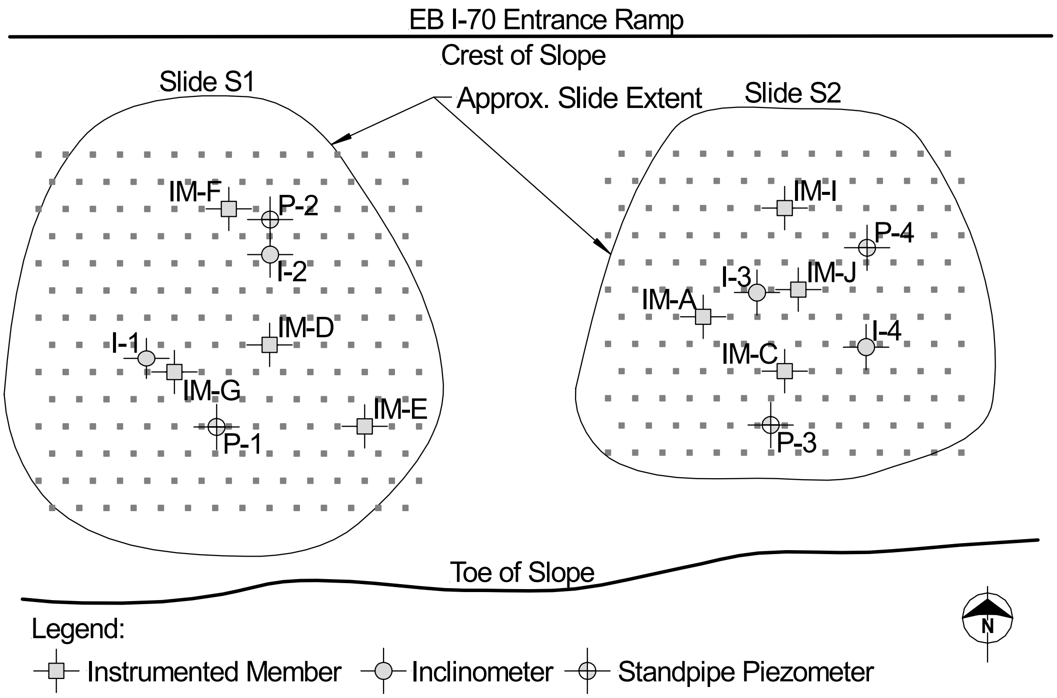


Figure 7.15 Plan view of slide areas S1 and S2 showing locations of instrumentation installed during Phase I at the I70-Emma site.

Ten of the recycled plastic members installed during Phase I were instrumented with 120-ohm electrical resistance strain gages (Vishay Measurements Group part number EP-08-500AF-120). The instrumented reinforcing members were similar to those used at subsequent test sites (Figure 5.15), except that the force-sensing resistors (FSR) were not used and four pairs of "shear gages" were installed on the sides of the members. The instrumented members installed during Phase I also differed from those installed in Phase II in that they were not outfitted with the connections needed to take readings using the data acquisition system subsequently developed in Phase II. Rather, each individual gage was measured by manually connecting bare-ended wires to a Vishay Measurements Group P-3500 "manual" readout unit, which displayed the strain reading for subsequent recording in a log book. As shown in Figure 7.15, four instrumented members, denoted members IM-D, IM-E, IM-F, and IM-G, were installed in slide area S1. Instrumented members IM-A, IM-C, IM-I, and IM-J were similarly installed in slide area S2. Two additional instrumented members, IM-B and IM-H, were installed within slide area S3 (the control slide during Phase I) to monitor the "free-field" behavior of the reinforcing members.

Five slope inclinometers were also installed at the site to monitor deformations in the stabilized areas (S1 and S2) and control section S3. Inclinometer casings installed during Phase I were placed in 4-in (10-cm) diameter boreholes extending approximately 5-ft (1.5-m) below the toe of the slope and backfilled with concrete sand. Two casings were placed in each of the stabilized areas as shown in Figure 7.15. One casing was installed near the center of slide area S3 to monitor the control slide.

Five continuously screened standpipe piezometers were also installed at the site to depths extending approximately 5-ft (1.5-m) below the toe of the slope. Two piezometers were placed in each of the stabilized areas along a line roughly through the center of the areas; one additional piezometer was installed near the center of slide area S3. The piezometers were complemented with several jet-filled tensiometers installed to depths of up to 4-ft (1.2-m) in close proximity to the piezometers. No other moisture sensors were installed at the site during Phase I.

7.4.2. Instrumentation Installed During Phase II

Following installation of reinforcing members in slide area S3 during Phase II, additional instrumentation was installed at the site to monitor the performance of the newly stabilized area. Instrumentation installed at this time is generally similar to that installed at the other test sites during Phase II, which includes improvements to overcome some of the limitations of the instrumentation installed during Phase I. Figure 7.16 shows a plan view of slide area S3 indicating approximate locations of the instrumentation installed in slide area S3 during Phase II.

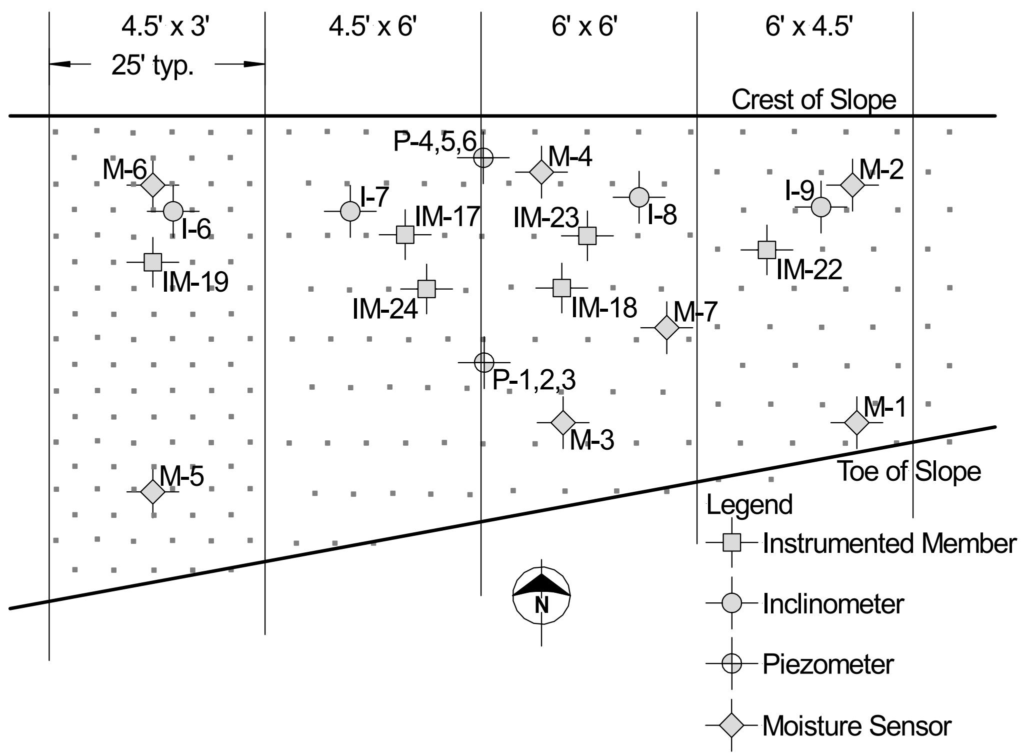


Figure 7.16 Plan view of slide area S3 at the I70-Emma site showing locations of instrumentation.

Six instrumented reinforcing members, similar to those described in Chapter 5 (Figure 5.15), were installed in slide area S3 during installation of all reinforcing members. All instrumented members were generally installed along a horizontal line passing just above the mid-point of the slope. Instrumented members IM-19 and IM-22 were installed in Section A and Section D, respectively, while two instrumented members were installed in Sections B (IM-17 and IM-24) and C (IM-18 and IM-23) to provide some redundancy in the sections that are most likely to fail.

Slope inclinometer casings were installed in each of the four reinforcement sections on January 27-28, 2003 in close proximity to the instrumented reinforcing members. The inclinometer casings were installed in 6-in (15-cm) diameter boreholes and backfilled with concrete sand. All casings were extended 19-ft (5.8-m) below grade to provide for adequate anchorage in stable strata.

Two clusters of standpipe piezometers similar to the one shown in Figure 5.24 were also installed in slide area S3 at the same time. Both clusters were installed along the slope section between Sections C and D and both contained three piezometers screened at different levels to permit possible perched water conditions to be detected. Piezometers P-1, P-2, and P-3 were placed in a cluster just below the center of the slide area and were screened at depths of 4.5-ft, 9.5-ft, and 14.5-ft (1.4-m, 2.9-m, and 4.4-m), respectively. Piezometers P-4, P-5, and P-6 were installed near the crest of the slope and screened at depths of 14.5-ft, 9.5-ft, and 4.5-ft (4.4-m, 2.9-m, and 1.4-m), respectively.

An array of moisture sensors similar to that installed at the I435-Wornall Road and US36-Stewartsville sites was also installed in slide area S3 in May 2003. Seven Profile Probe® access tubes were installed across the slide area at locations denoted M-1 through M-7. In addition, an array of two Thetaprobes® and two Equitensiometers® was installed at location M-7 to provide for essentially continuous monitoring of moisture conditions within the slope.

7.5. Field Performance

Instrumentation installed during Phase I in slide areas S1 and S2 has been monitored for 42 months. Instrumentation installed in slide area S3 during Phase II has been monitored for 6 months. In the following sections, the results obtained from instrumentation installed at the site during these periods are presented. Results from slide areas S1 and S2 are simply summarized to provide an update on the performance of these areas since Phase I was completed. More complete results are provided for slide area S3.

7.5.1. Precipitation at the I70-Emma Site

Figure 7.17 shows daily and monthly precipitation totals recorded at the Sedalia Memorial Airport weather station since November 1999. The Sedalia weather station is located approximately 25 miles southeast of the I70-Emma site. Normal precipitation patterns at the site consist of relatively wet spring seasons, moderate rainfall during the summer and fall months, and relatively dry winter seasons. This pattern has been generally observed since November 1999, although overall rainfall totals have varied from year to year. Precipitation between November 1999 and May 2000 was much lower than normal. Precipitation increased in June 2000 and remained at or above normal through the winter months. Precipitation in spring 2001 was then much higher than normal with several consecutive months where precipitation exceeded 5-in/month. Heavy precipitation in spring 2001 resulted in failure of both control slides during Phase I; slide area S4 failed in late April 2001 and slide area S3 failed on June 5-6, 2001. Aside from several notable deviations, precipitation since that time has been at levels near or slightly below normal. Notable deviations from normal precipitation include a very dry period during May and early June of 2002, a relatively wet period in October and November 2002, and a relatively dry period during summer 2003.

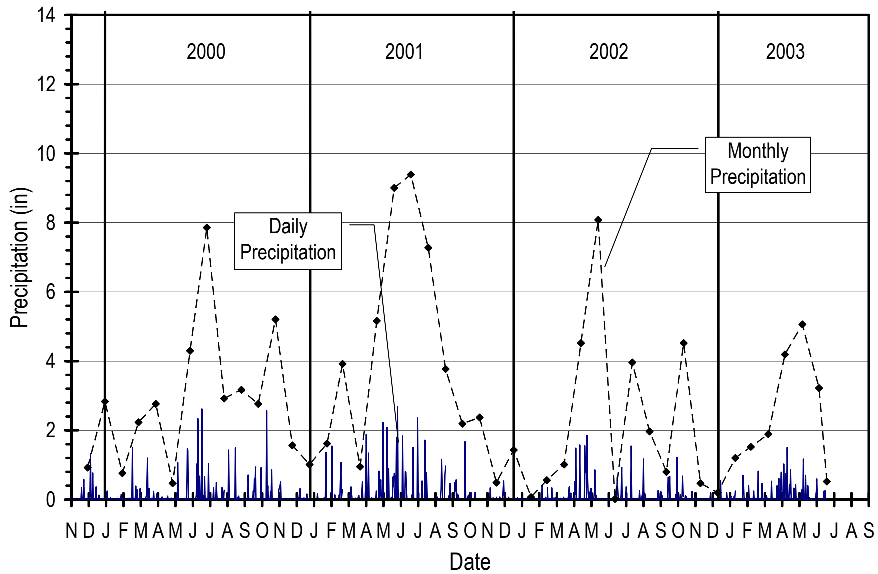


Figure 7.17 Daily and monthly precipitation from Sedalia Memorial Airport weather station.

7.5.2. Performance of Slide Areas S1, S2, and S3 During Phase I

Piezometers and tensiometers. The performance of slide areas S1 and S2 has been monitored for over three years. Unfortunately, the continuously screened piezometers and manual tensiometers installed in these areas have not provided reliable data regarding the moisture conditions within the slope. The continuously screened piezometers have indicated some water is present within the slope. However, because the piezometers are continuously screened, it has been difficult to interpret the source of this water and the associated pore pressure conditions. The jet-filled tensiometers have indicated variable soil suctions in response to precipitation at the site. However, since the tensiometers were only monitored at discrete intervals, it has been extremely difficult to correlate the field readings with precipitation events. The tensiometers have also become damaged on several occasions due to freezing temperatures. Results from these instruments have therefore not been terribly useful for developing an understanding of the pore pressure conditions within the slope to date. It is hoped that additional instrumentation installed during Phase II, which will provide better piezometer data and essentially continuous measurements of soil water content and soil suction, will provide much better data to evaluate performance in these slide areas in the future.

Inclinometers. Figure 7.18 shows the lateral deflection profile determined from inclinometer I-2 in slide area S1 at the I70-Emma site. Other inclinometers showed generally similar profiles. However, some problems were experienced with some inclinometers indicating significant "up-slope" movements at depth. These "movements" are attributed to movement of the casing within the borehole as a result of inadequate backfilling of the casings rather than actual movements in the slope. As a result of these problems, inclinometer measurements have been somewhat scattered over time. As shown in the figure, movements have generally been greatest near the ground surface with continuously decreasing movements with depth. This trend was consistent among the remaining inclinometers, although the magnitudes of the overall movements varied somewhat.

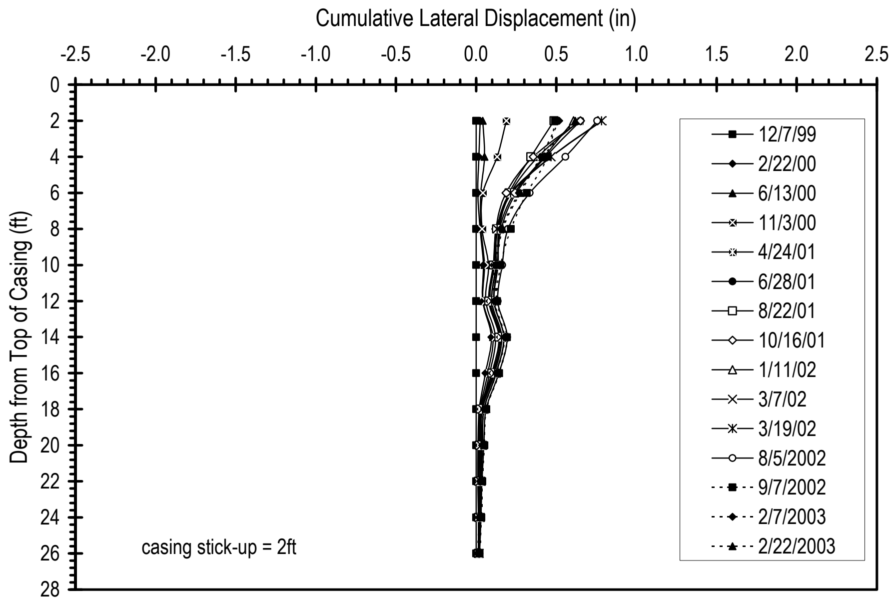


Figure 7.18 Lateral deflection profile for inclinometer I-2 in slide area S1 at I70-Emma site.

Figure 7.19 shows the cumulative deformations determined from each of the inclinometers installed at the site versus time. Although these data are more sporadic, the trend in behavior is generally consistent with the behavior observed at the I435-Wornall Road site. Movements were generally small during most of the first year following installation. This is consistent with the lower than normal precipitation experienced at the site during this time. In late 2000 through May 2001, however, movements were observed to increase significantly in response to increased precipitation. Lateral movements have since remained relatively constant, presumably due to the fact that resistance in the reinforcing members became mobilized. Inclinometers I-1, I-3, and I-4 all indicated a significant "jump" in lateral deformations between February and June 2002 during a period of increased precipitation. However, this increase in deformations follows a period of apparently up-slope deformations between August 2001 and February 2002, which may simply be scatter in the measurements or movement within the borehole since upslope movements are not considered realistic. This possibility is supported by readings from inclinometer I-2, which has been relatively stable during the same period. Regardless of the reasons for the apparent "up-slope" movements followed by "down-slope" movements, the overall movements indicated by all of the inclinometers in early 2003 are similar in magnitude to the overall movements indicated in August 2001, which suggests that significant movements have not occurred since that time. The magnitudes of overall movements vary from approximately 0.5-in (1.3-cm) to 1.5-in (3.8-cm) with inclinometers I-1 and I-2 in slide area S1 having movements between

0.5- and 1.0-in (1.3- and 2.5-cm) and inclinometers I-3 and I-4 in slide area S2 indicating movements between 1.0- and 1.5-in (2.5- and 3.8-cm).

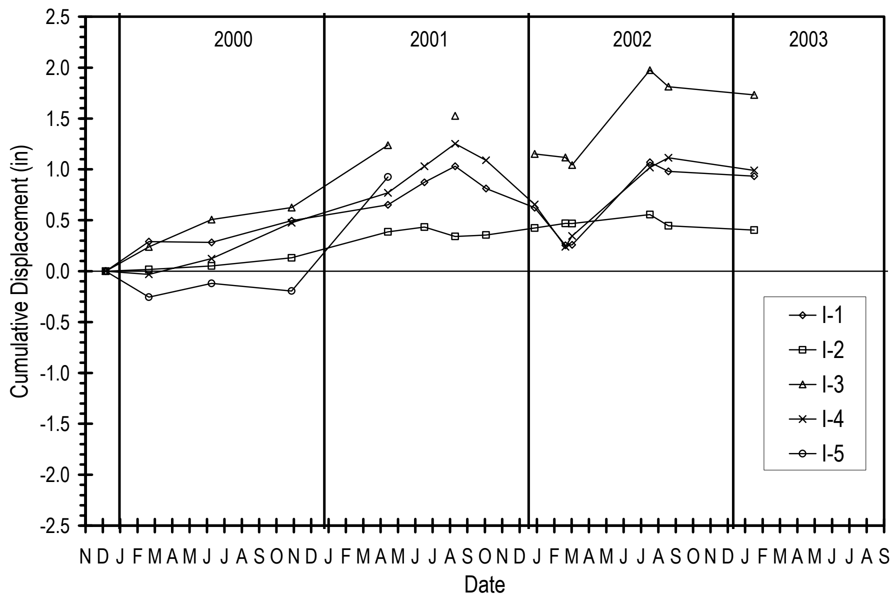


Figure 7.19 Cumulative lateral deflections for inclinometers I-1 through I-5 at depths between 3- to 4-ft for slide areas S1 and S2 at the I70-Emma test site.

Inclinometer I-5 is somewhat of a special case. This inclinometer was installed in slide area S3, which served as a control slide during Phase I. Control slide S3 failed on June 5-6, 2001. The failure occurred just above the location where inclinometer I-5 met the ground surface and the failure "kinked" the inclinometer casing so that no further readings could be taken. Figure 7.19 shows that this inclinometer indicated negligible movements over the first year following installation. However, movements indicated by the inclinometer then increased dramatically over the next few months in response to increased rainfall. The inclinometer was then rendered inoperable in June 2001, when the control slide failed. This pattern of deformation is generally consistent with what would be expected for the observed rainfall patterns and is indicative of the failure. The fact that the remaining inclinometers in the stabilized sections have indicated little additional movements supports the observation that the movements experienced in the stabilized sections were simply movements required to mobilize the resistance in the reinforcing members.

Instrumented reinforcing members. Readings have been taken on all instrumented reinforcing members in slide areas S1, S2, and S3 since installation was completed in December 1999. These data have been reduced and interpreted to establish the magnitudes of axial stresses and bending moments in the reinforcing members since installation. No initial readings were taken for the reinforcing members prior to installation during Phase I. The results presented here are therefore "incremental" stresses and bending moments induced since installation as discussed in Section 5.5.2. The most reliable data has been obtained from instrumented member IM-G from slide area S1, member IM-C from slide area S2, and member IM-H from slide area S3 (a single member in the control area). The results presented below are therefore for these members.

Figure 7.20 shows the distribution of axial stresses determined for instrumented member IM-G. The observed distribution is generally parabolic with negligible stresses near the two ends of the member and the maximum axial stress near the midpoint of the member. Members IM-C and IM-H had similar distributions of axial stress although the magnitudes of the stresses differ.

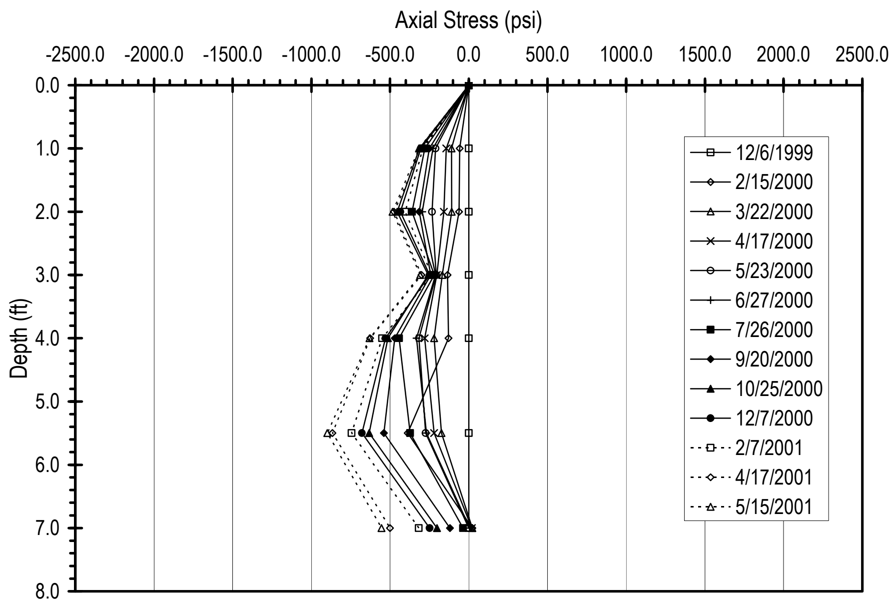


Figure 7.20 Measured incremental axial stress in instrumented member IM-G in slide area S1 at I70-Emma site during Phase I.

Figure 7.21 shows the maximum incremental axial stresses determined for instrumented members IM-G, IM-C, and IM-H plotted as a function of time. As has been consistently observed at other sites, the incremental stresses are all negative, which indicates tensile stresses/strains since installation. This is again attributed to relaxation of compressive stresses induced in the members since installation, and may in fact only indicate negative *changes* in stress as opposed to actual tensile stresses.

It is interesting to note however that member IM-C, which was installed with a vertical orientation in slide area S2, experienced a slight decrease in stress over the first year after installation after which the axial stress has stayed relatively constant. Member IM-G, which was installed roughly perpendicular to the face of the slope, also experienced a gradual decrease in stress over this time, but the magnitude of the incremental stress is much greater. It is believed that the incremental axial stresses developed in member IM-C were significantly lower than member IM-G because slope movements parallel to the face of the slope would tend to resist any axial relaxation for members installed vertically, while slope movements for members installed perpendicular to the slope would not tend to influence the axial stresses. Unfortunately, member IM-G was rendered inoperable in late May 2001 so it is impossible to determine whether the incremental axial stresses in the member have stabilized. Member IM-H, installed vertically in control slide area S3, behaved similar to member IM-C (also installed vertically) over the first year following installation. However, the incremental axial stresses in IM-H then decreased substantially prior to the failure of slide area S3 on June 5, 2001. This rapid decrease in the incremental axial stresses occurs at the same time as movements in slide area S3 began to accelerate and is believed to be a result of the slope beginning to fail.

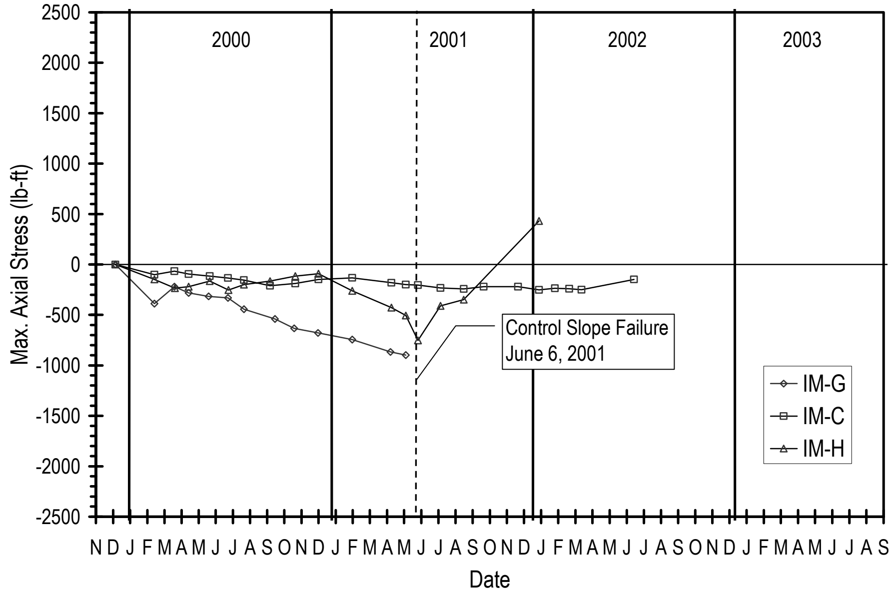


Figure 7.21 Maximum axial stress in instrumented members IM-G, IM-C, and IM-H at I70-Emma test site during Phase I.

Figure 7.22 shows the distribution of bending moments determined for instrumented member IM-H. The distribution is also generally parabolic with negligible moments near the ends of the member and the maximum moment occurring near the midpoint of the member. Moments are generally positive along the entire length of the member. Moment distributions for members IM-G and IM-C were similar in shape but with generally lower magnitudes.

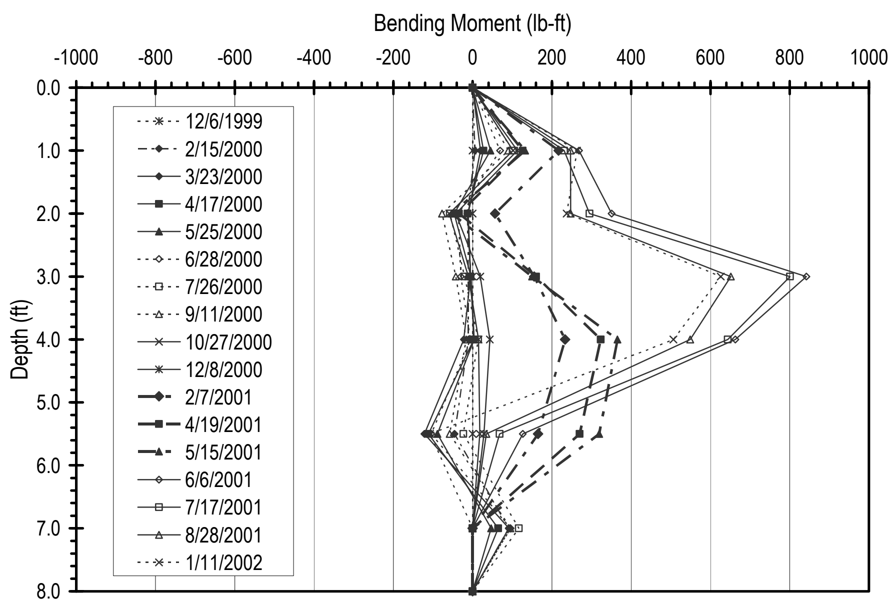


Figure 7.22 Measured bending moments in instrumented member IM-H at I70-Emma test site during Phase I.

Figure 7.23 shows the maximum moments determined for these three members plotted as a function of time. Member IM-C experienced gradually increasing moments during the first 17 months following installation. Then, around the time of the failure of control slide S3, the moments in IM-C increased by a small but noticeable amount. This sudden increase is believed to be a response to the slope needing additional resistance to maintain equilibrium at the time of the control slide failure. Incremental bending moments since that time have remained essentially constant despite having several periods of heavy rainfall during that time.

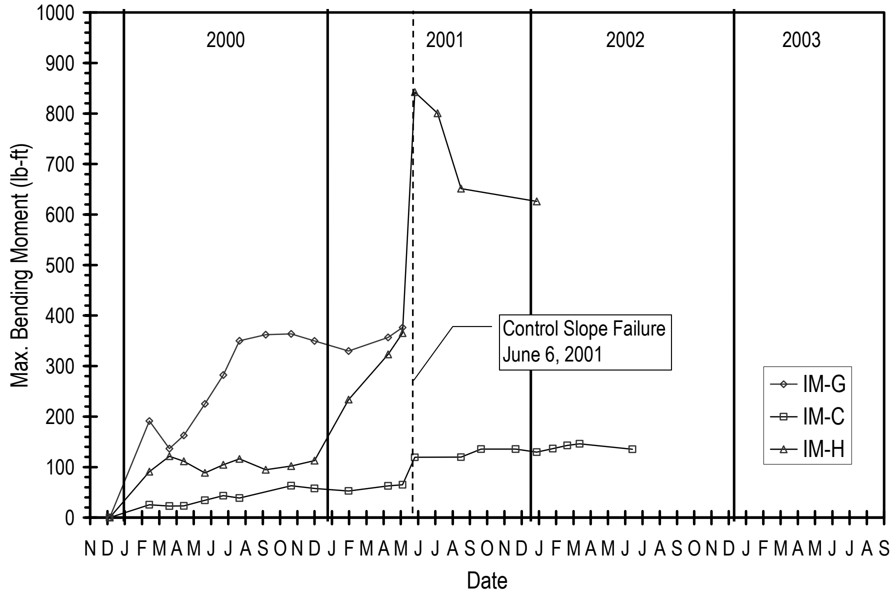


Figure 7.23 Maximum bending moments in instrumented members IM-G, IM-C, and IM-H at I70-Emma test site during Phase I.

In contrast, Member IM-G experienced a relatively rapid increase in the incremental bending moments during the first 8 months following installation after which the incremental bending moments remained essentially constant until, during the three months prior to the failure of the control slopes, the bending moments increased slightly. Unfortunately, IM-G became inoperable just prior to the failure of the control slide. However, the slight increase in bending moments leading up to this time indicates that the reinforcing members were providing additional resistance needed to maintain the equilibrium of the slope. While the incremental bending moments in member IM-G just after the control slide failure are not known, the fact that inclinometers IM-1 and IM-2 have not shown any significant movement since that time suggests that the bending moments would have remained essentially constant. Member IM-H indicated behavior similar to member IM-C during the first year following installation, although the member had slightly higher bending moments. However, incremental bending moments in member IM-H increased steadily during the 6 months leading up the failure of the control slide, which appear to be a response to the slope needing additional resistance. Readings taken on member IM-H on the day after the failure indicated that a significant increase in bending moments had occurred. This dramatic increase is believed to be in response to the failure of the slope. The magnitude of the bending moments measured just after the failure is approximately 850-lb-ft (1200-N-m), a value which is relatively close to the nominal moment capacity of the recycled plastic members (~1000-lb-ft). Member IM-H was subsequently exhumed in early 2002, when it was determined that the member had fractured at a distance of approximately 5-ft (1.5-m) below the top of the member.

It is interesting to compare the behavior of member IM-G, which was installed perpendicular to the face of the slope, with member IM-C which was installed vertically. Results determined from the field instrumentation indicates that member IM-G experienced greater incremental axial stresses and greater incremental bending moments than member IM-C which was installed vertically. Furthermore, prior to the months leading up to the failure of the control slide, member IM-H, which was also installed vertically, exhibited behavior similar to member IM-C. This evidence suggests that members installed perpendicular to the slope will be subjected to noticeably higher bending moments since slope movements will directly contribute to these moments. In contrast, members installed vertically are subjected to lower incremental bending moments since down-slope movements will tend to produce a compressive axial stress in addition to a bending moment. This postulated load transfer is further supported by the measured axial stresses in the members. Recalling the hypothesis that the observed incremental axial stresses/strain are in fact a result of relaxation of compressive stresses/strains developed during installation, it can be noted that *incremental* axial stresses for members IM-C and IM-H were of smaller magnitude (i.e. less relaxation of stresses) than was observed for member IM-G which was installed perpendicular to the slope and therefore would not experience a significant compressive load as the soil moves down-slope. This postulated load transfer is consistent with that observed at other field test sites.

7.5.3. Performance of Slide Area S3 During Phase II

Piezometers and moisture sensors. Water levels measures in the piezometers installed in slide area S3 during Phase II are plotted in Figure 7.24. Piezometers P-1 and P-6, both screened at a depth of 4-ft (1.2-m) below the ground surface, have been dry since the piezometers were installed on January 27-28, 2003. The remaining piezometers all experienced rising water levels between January and late May 2003, which is consistent with the increasing precipitation that occurred over this period. Piezometers P-2 and P-5, both of which are screened at approximately 9-ft (2.7-m) below ground surface, have shown similar water levels during the monitoring period ranging from 7- to 9-ft (2.1- to 2.7-m). The deepest piezometers, P-3 and P-4, show differing water levels with P-3 indicating water levels near the ground surface and P-4 indicating water levels between 12- and 14-ft (3.7- and 4.3-m).

Measurements taken for the soil moisture and soil suction sensors located at location M-7 near the midpoint of the slope are plotted in Figure 7.25. These sensors were not installed until late May 2003 so there is no indication of moisture levels or pore pressures prior to this date. Readings during June 2003 indicate relative stable moisture content and pore pressures while readings since June indicate rapidly dropping pore pressures and moisture contents in all sensors in response to the lack of precipitation experienced during July 2003.

Slope inclinometers. Lateral deflections measured for inclinometer I-7 at the I70-Emma site are plotted in Figure 7.26. The deflection profile indicated by these measurements is consistent with those measured for other inclinometers at the site, including those installed during Phase I. The profile indicates that the maximum lateral deformations are occurring at the ground surface with continuously decreasing deformations with depth. Maximum deformations for the inclinometers installed in each of the four test sections during Phase II are plotted versus time in Figure 7.27. This figure indicates that lateral movements in each of the sections have been small. The smallest movements have occurred in Sections A (I-6) and D (I-9), which are the sections with the highest concentrations of reinforcing members. Movements in Sections B and C have been somewhat larger and appear to have increased somewhat in response to increased precipitation in April and May 2003. Overall, however, the movements remain much smaller than the movements that have been required to mobilize significant resistance in the reinforcing members at previous sites.

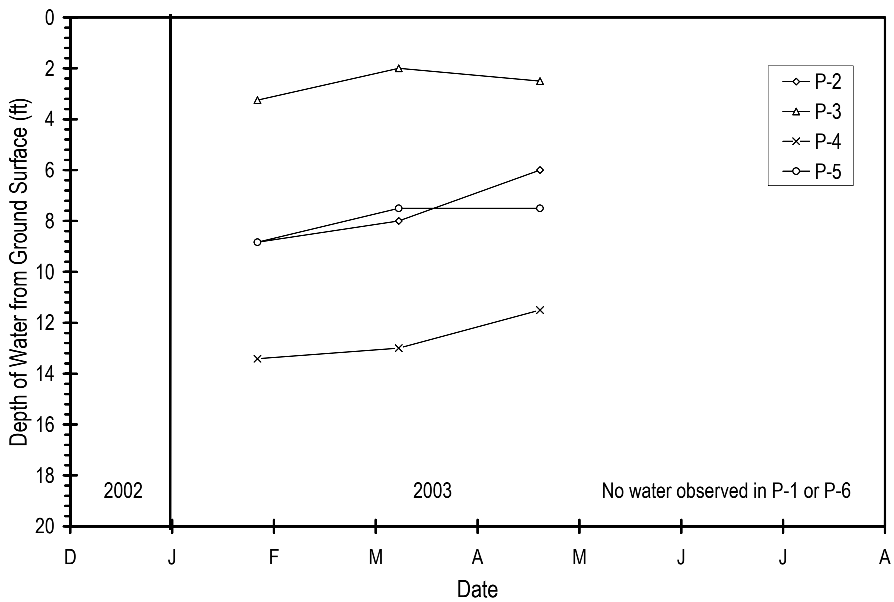
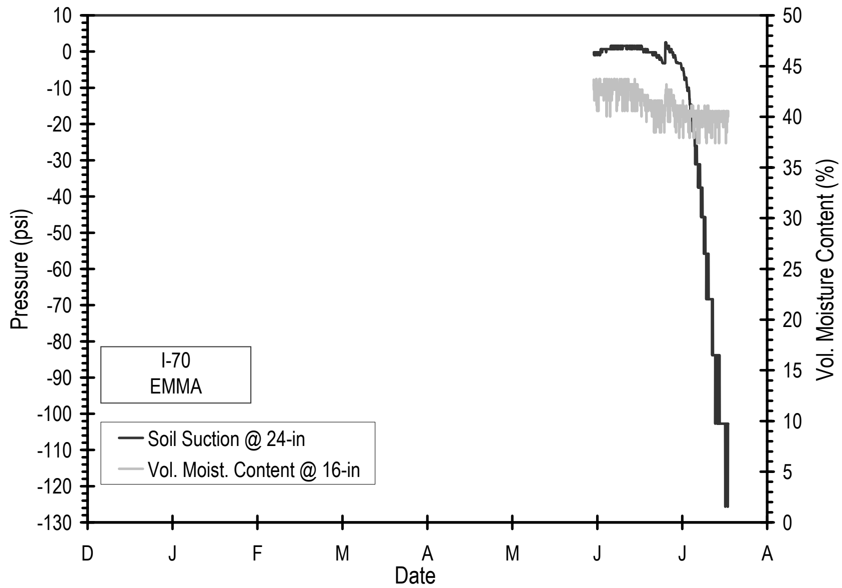


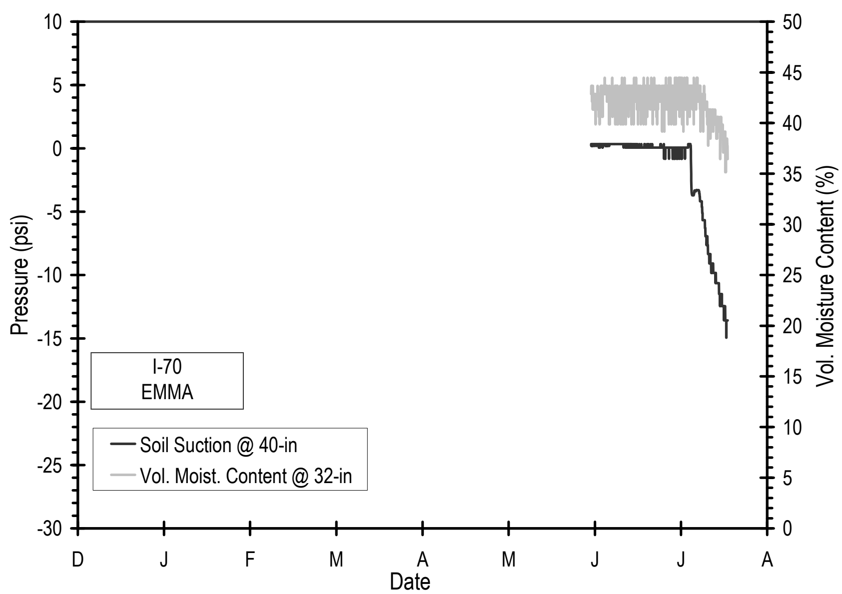
Figure 7.24 Piezometric water levels measured at I70-Emma test site.

Instrumented reinforcing members. Incremental and overall axial stresses measured for instrumented member IM-24 are plotted in Figure 7.28. As shown in the figure, the distribution of axial stresses is generally parabolic in shape with negligible stresses near the top and tip of the reinforcing member with the maximum axial stress occurring near the midpoint of the member. Instrumented members IM-18, IM-22, and IM-23 had similar distributions of axial stresses. Members IM-17 and IM-19 had axial stress distributions that generally increased with depth, as has been observed in instrumented members at other sites.

The maximum axial stresses measured for each member are plotted as a function of time in Figure 7.29. As has been observed at all other test sites, the trend in incremental axial stresses is to have increasingly negative incremental stresses with time, which is believed to be a relaxation of stresses imposed on the members during installation. Overall axial stresses measured during Phase II have generally been small and most have been determined to be tensile stresses. The exceptions to this observation are members IM-17 and IM-18, which both show initially compressive overall stresses but more recently have indicated tensile stresses, and member IM-23, which shows relatively high compressive stresses just after installation. As stated in previous chapters, there is some question as to whether the true magnitude of the initial stresses imposed during installation was accurately captured by the strain gages. The magnitudes of the overall stresses are therefore somewhat questionable, but the trend of having decreasing incremental stresses with time has been consistently observed at all sites.



(a) readings for shallow sensors



(b) readings for deeper sensors

Figure 7.25 Soil suction and volumetric water content from I70-Emma test site.

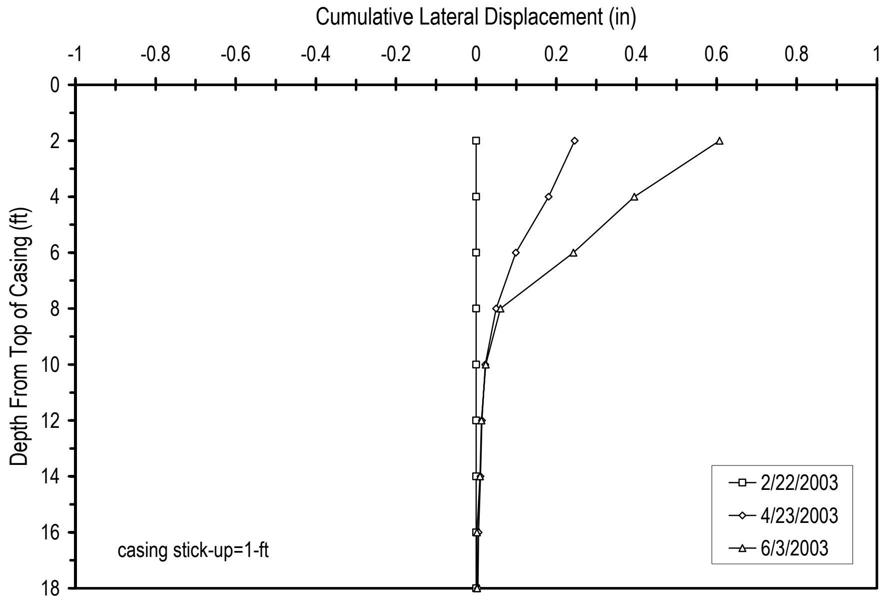


Figure 7.26 Lateral deflection profile for inclinometer I-7 at I70-Emma site.

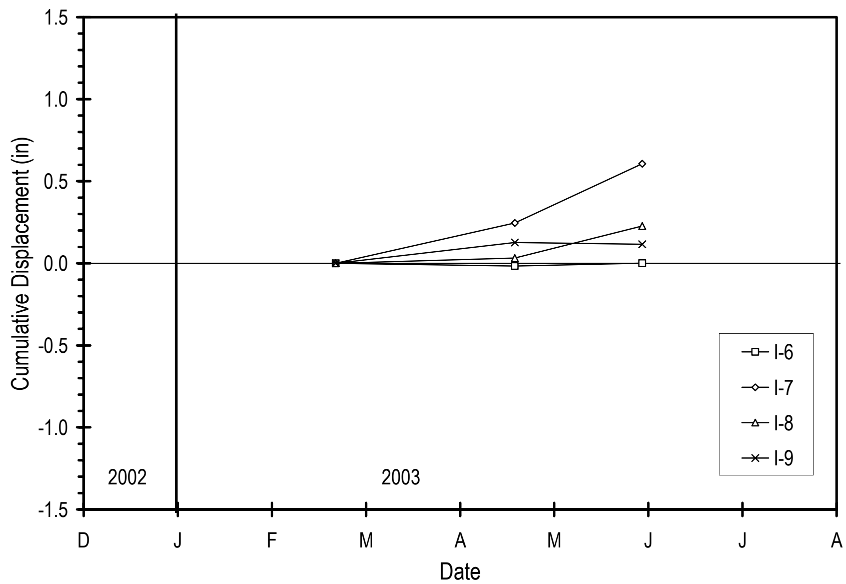
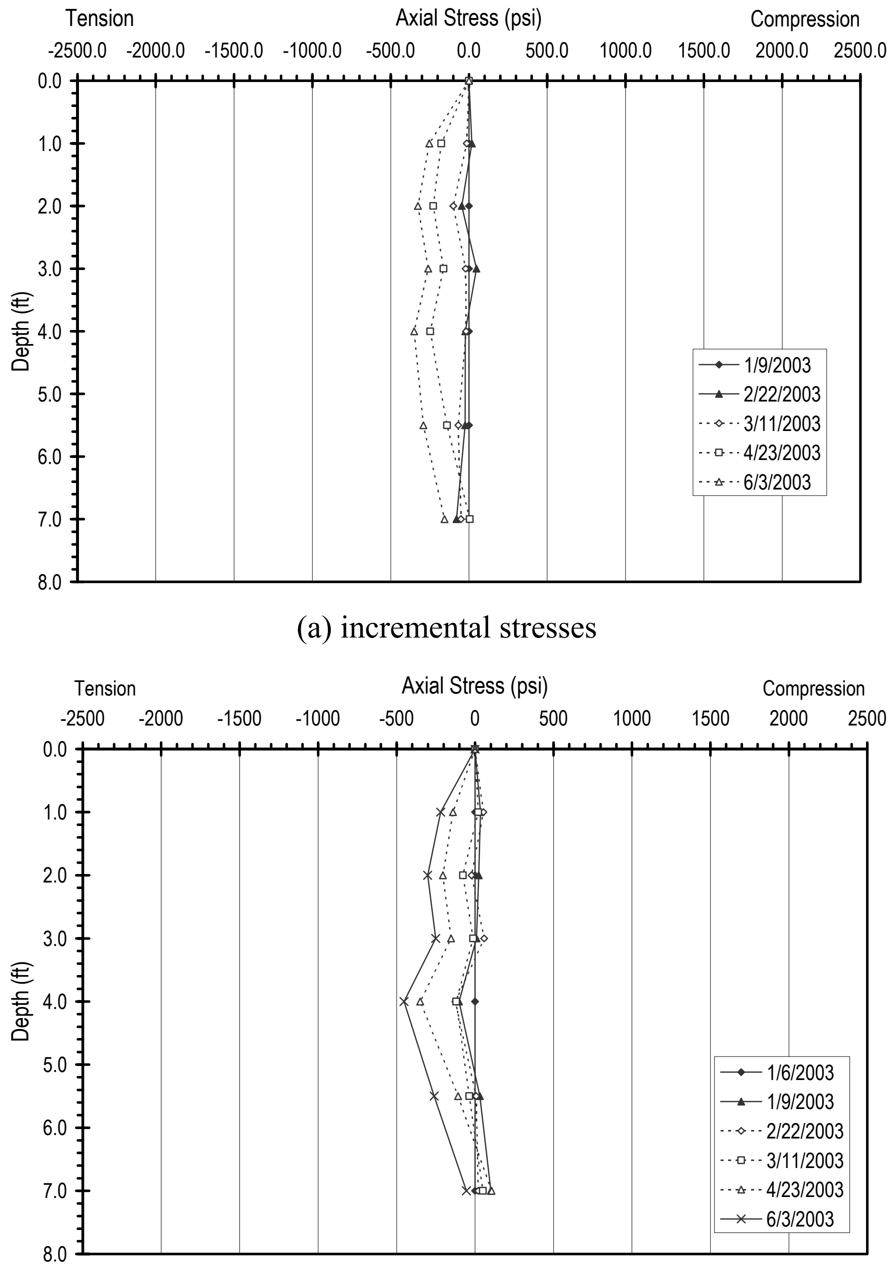


Figure 7.27 Cumulative lateral deflections at depth of 1-ft for inclinometers I-6 through I-9 at I70-Emma test site.

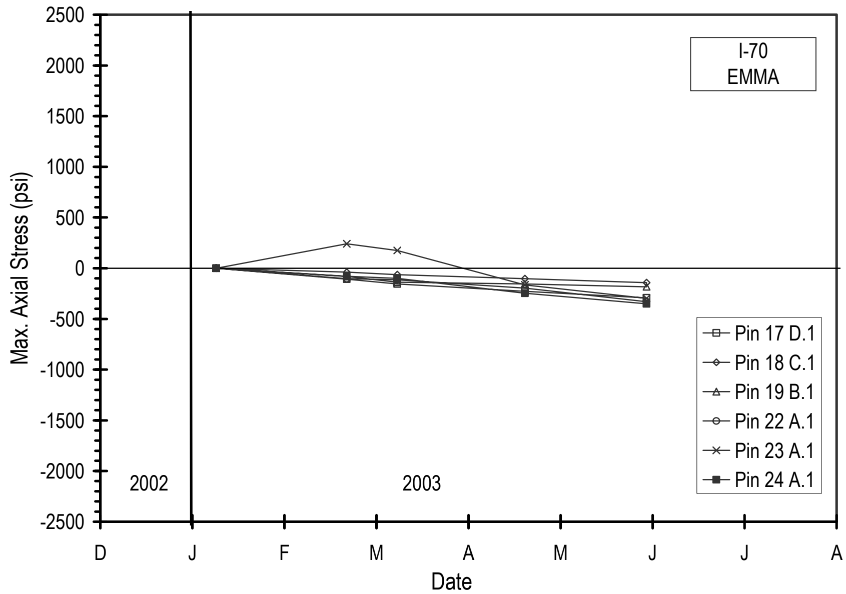
Profiles of the bending moments measured at the I70-Emma test site during Phase II have taken two general forms. The first form is a generally parabolic distribution of moments with negligible moments at the ends of the member and the maximum moment near the midpoint of the member as shown in Figure 7.30 for instrumented member IM-22. Member IM-19 had a similar moment distribution. The second form of the distribution of moments along the reinforcing member is an S-shaped distribution as shown in Figure 7.31 for instrumented member IM-18. In this distribution, bending moments in the upper portion of the member are generally positive while bending moments in the lower portion of the member are generally negative. Members IM-17, IM-23, and IM-24 also exhibited this type of distribution.



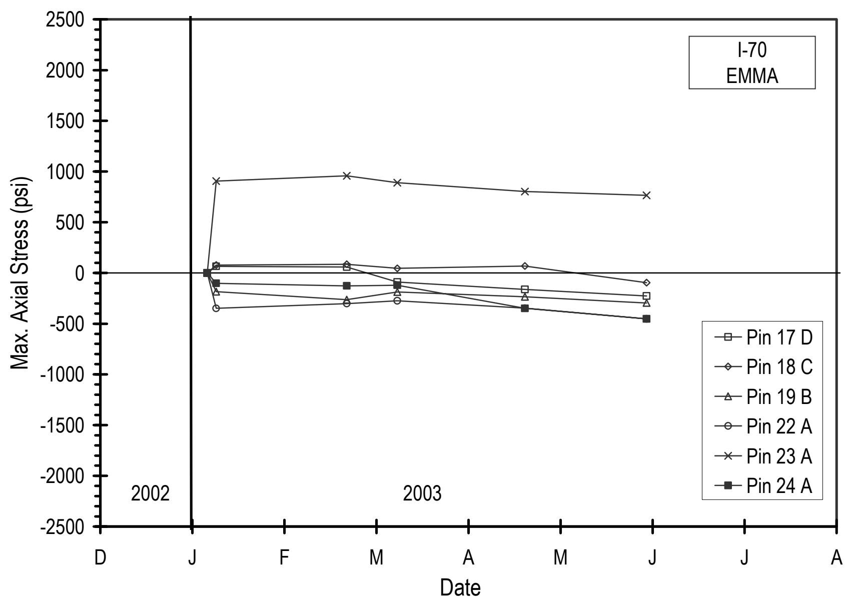
(a) incremental stresses

(b) overall stresses

Figure 7.28 Measured axial stresses for instrumented member IM-24 in slide area S3 at the I70-Emma test site: (a) incremental stresses and (b) overall stresses.



(a) incremental axial stresses



(b) overall axial stresses

Figure 7.29 Maximum axial stresses in instrumented members at I70-Emma test site during Phase II: (a) incremental stresses and (b) overall stresses.

The maximum incremental and overall bending moments for each of the instrumented members installed at the I70-Emma test site during Phase II are plotted versus time in Figure 7.32. Aside from member IM-23, incremental bending moments for all members have increased gradually since installation. Incremental bending moments for instrumented member IM-23 have increased significantly between the first and second readings and then decreased since that time. This response is not generally consistent with the response of the other members, including member IM-24 which is installed in the same test section, so the results for IM-23 are questionable. Figure 7.32b indicates that significant bending moments were imposed in each of the instrumented members during installation. Since that time, the overall bending moments have remained relatively constant, although members IM-22, IM-23, and IM-24 have experienced slight increases in the overall bending moments between February and April 2003. The overall magnitudes of the bending moments for all instrumented members remain well below the moment capacity of the reinforcing members (approx. 1000-lb-ft).

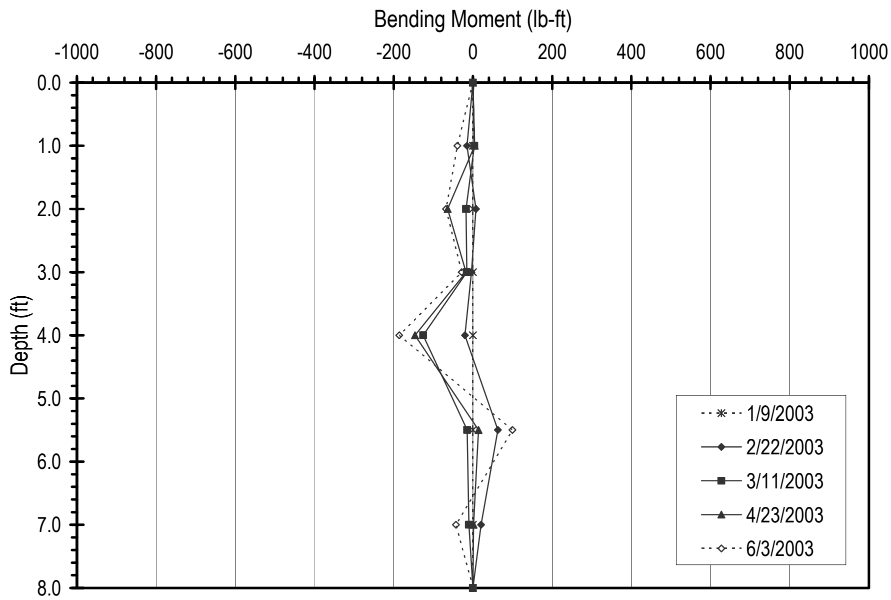


Figure 7.30 Measured incremental bending moments for instrumented member IM-22 at I70-Emma test site during Phase II.

Overall, results obtained from the field instrumentation installed during Phase II indicates that the reinforcing members have yet to provide significant resistance to maintain the stability of the slope. This observation is supported by several facts presented above. First, while precipitation has increased since installation was completed, the overall level of precipitation is significantly lower than is normally experienced at the site. Measurements of pore pressures from the piezometers and soil moisture and soil suction sensors in recent months indicate that the slope has remained relatively dry since installation, which suggests that the slope would likely have been stable during this time without the reinforcing members. Secondly, readings from the slope inclinometers indicate that only very small lateral movements have occurred in the stabilized area. These movements are not believed to be sufficient to mobilize significant resistance in the reinforcing members based on results from other sites, which further supports the belief that the slope would have been stable over the monitoring period without the reinforcing members. Finally, measurements made from the instrumented reinforcing members indicates that the current axial stresses and bending moments are essentially identical to those estimated following installation, which indicates that additional resistance required to maintain the stability of the slope has not been mobilized (and by inference, not been needed). While the resistance provided by the reinforcing members to date is small, it is expected that the mobilized resistance will increase substantially during future periods with average to above average rainfall and it is critical that monitoring of the field instrumentation be continued through that time.

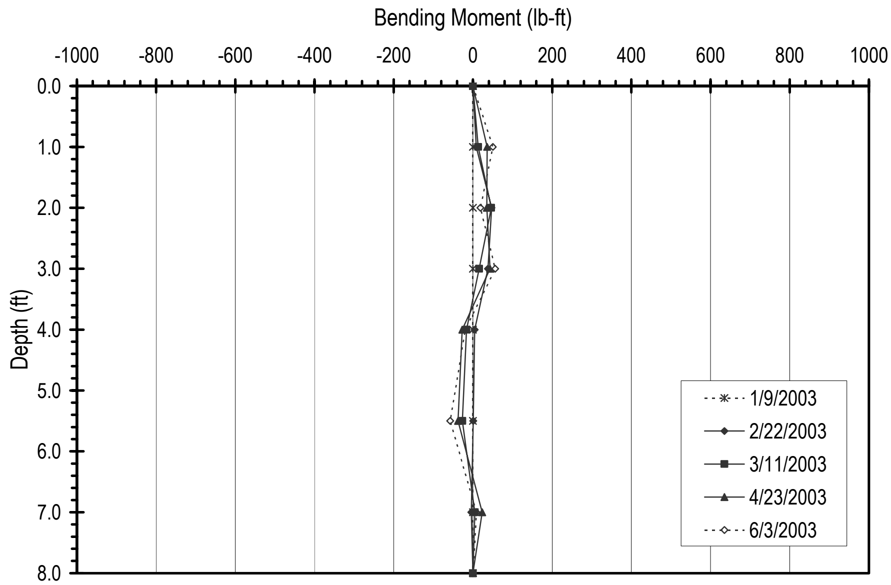
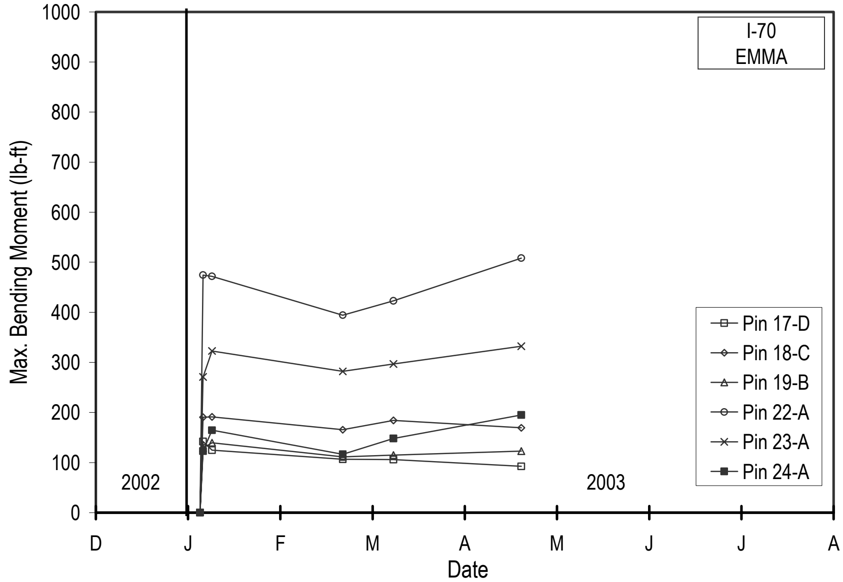


Figure 7.31 Measured incremental bending moments for instrumented member IM-18 at I70-Emma test site during Phase II.

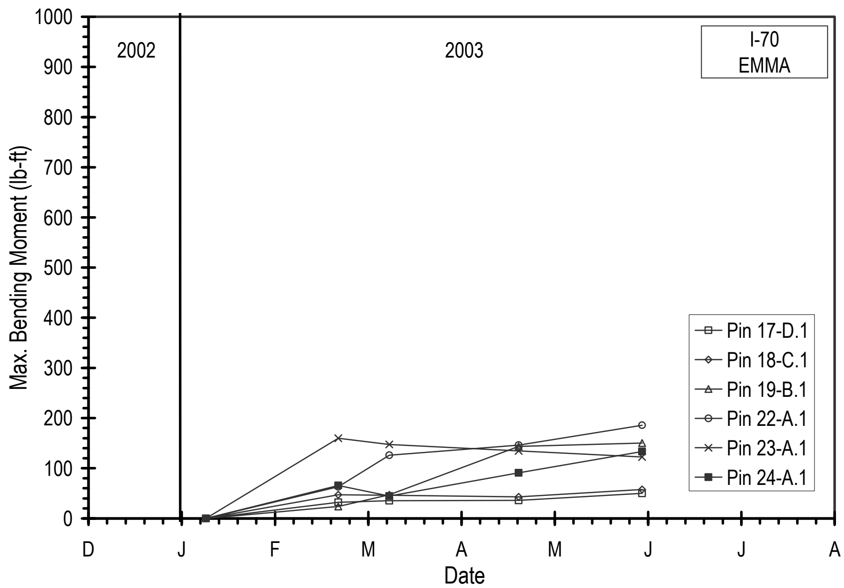
7.6. Status of Instrumentation at I70-Emma Test Site

The instrumentation at the I70-Emma test site is in various states of repair. All inclinometers except for inclinometer I-5, which was located in slide area S3 during Phase I and was rendered inoperable when the control slide failed, continue to be in good working order. Regular readings continue to be taken for all remaining inclinometers. Piezometers and tensiometers installed during Phase I have not provided information that is of significant benefit for interpreting the response of slide areas S1 and S2. Monitoring of these devices has therefore been discontinued. However, additional soil moisture and soil suction sensors installed during Phase II are expected to allow much better information regarding the pore pressure conditions within all three slide areas to be obtained. Piezometers installed in slide area S3 during Phase II are also expected to contribute in this regard.

The instrumented reinforcing members installed during Phase I have deteriorated to the point where additional readings will be of little value. As noted above, member IM-G was damaged in May 2001. Member IM-H was also damaged during the control slide failure and was subsequently removed for inspection. Many of the strain gages for the remaining instrumented members installed during Phase I have also become inoperable. As a result of this, and because taking readings for members installed during Phase I is a very labor intensive process, additional readings for the instrumented members installed during Phase I have been discontinued at this time. In contrast, instrumented reinforcing members installed in slide area S3 continue to perform well. Readings for these members will continue to be taken for the foreseeable future with the hope that reliable readings can be made when the members begin to mobilize significant resistance to maintain the stability of the slope.



(a) incremental bending moments



(b) overall bending moments

Figure 7.32 Maximum bending moments in instrumented members at I70-Emma test site during Phase II: (a) incremental bending moments and (b) overall bending moments.

7.7. Summary

The activities performed to establish three separate test areas at the I70-Emma test site were described in this chapter. The site includes four separate slide areas that have experienced repeated failures in the past. Two of these slide areas were stabilized during Phase I while the remaining two slide areas were used as control sections. Both control sections subsequently failed in spring 2001 and one of the former control slide areas was subsequently selected for stabilization during Phase II. In this chapter, the general characteristics of the site were described. Activities undertaken to select and construct the stabilized sections were then summarized. Finally, the results obtained from monitoring the performance of the three stabilized areas since installation were presented.

Chapter 8. US54-Fulton Site

The final site selected for stabilization during Phase II is the US54-Fulton test site. The site is located on U.S. Highway 54 approximately 2 miles north of Fulton Missouri, just south of Richland Creek. Activities undertaken to establish the test site are described in this chapter along with results of field performance monitoring activities performed to date.

8.1. Site Characteristics

The slope at the US54-Fulton site is an excavated slope constructed for the approach to the nearby bridge across Richland Creek. The slope is approximately 46-ft (14-m) high at its highest point with an inclination of approximately 3.2H:1V. Figure 8.1 shows an air photo of the area indicating the location of the slope. Prior to being selected for stabilization, the slope experienced a large surficial slide shown in Figure 8.2 that dammed the surface drainage features alongside U.S 54. The slide involved approximately 275-ft (85-m) of the slope measured parallel to US54 and was confined to the lower two-thirds of the slope. The slide appeared to be a retrogressive slide with the primary slide involving the lower half of the slope after which the upper portion of the slope subsequently failed.

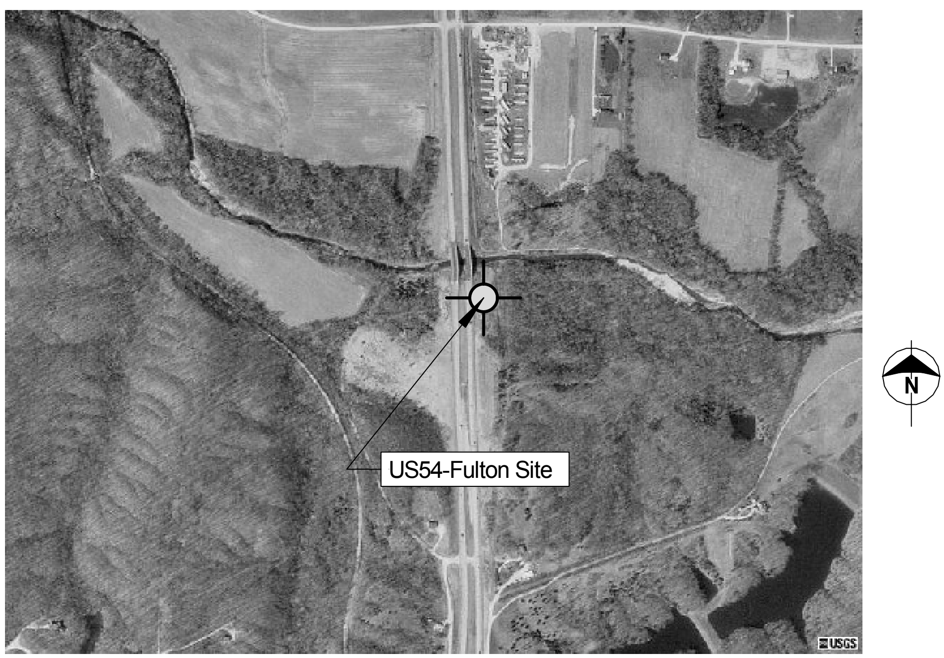


Figure 8.1 Air photo of area surrounding US54-Fulton site taken April 3, 1995 showing location of slope selected for stabilization (from USGS).

Boring and sampling for the US54 site took place during the period September 25-October 11, 2000. A total of 12 borings were placed throughout the area of the slide and just to the south of the slide area. A plan view of the site indicating the locations of all borings is provided in Appendix D along with logs of all borings. Continuous 3-inch (7.6-cm) diameter Shelby tube samples were taken in eight of the borings where soil conditions permitted. Where good quality Shelby tube samples could not be acquired, a Standard sampler was used to acquire disturbed samples for classification testing. In the remaining four borings, Standard Penetration Tests (SPT) were performed at 18-in (45-mm) intervals. The boring logs and subsequent laboratory testing indicate the soil profile at the US 54 site generally consists of lean to fat clay of variable stiffness with traces of sand and gravel found throughout the depths investigated. The clays are believed to be of glacial origin (ablation till). Fissures were observed in several of the borings at depths exceeding 10-feet (3-m), which indicates that sliding has previously occurred in the clay materials. Gypsum crystallizations were observed in several of these fissures. While the precise origin of these fissures is unknown, they are not believed to be associated with the current slide as they are located at depths that are not consistent with the observed features of the slide. Bedrock was not encountered in any of the borings at the US54 site. Standard Penetration Test (SPT) N_{60}-values measured in the soils less than 8-ft (2.4-m) in depth ranged from 1 to 23, with most values being between 8 and 10. N_{60}-values at greater depths ranged from 10 to 35 with most values in the range of 15 to 25.

Figure 8.2 Photograph of US54-Fulton slope following most recent slide event.

Laboratory testing of samples from the US54 site again consisted of natural moisture content tests, Atterberg limit tests, and consolidated-undrained type triaxial tests with pore pressure measurements. Measured moisture contents ranged from 10 to 35 percent. Samples taken from significant depths tended to have relatively consistent moisture contents of approximately 18 to 20 percent, while samples taken from near the surface tended to have highly variable moisture contents. There was some tendency for surficial samples taken from outside the slide area to have lower moisture contents than samples taken from within the slide area, although this observation was not universal.

Results of Atterberg limits tests indicated a general trend of increasing liquid limit (LL) and plasticity index (PI) with depth with essentially constant plastic limits (PL) for all specimens. Liquid limits for soils taken from depths less than 8-ft (2.4-m) ranged from 30 to 45, while LL for samples from greater depths ranged from 40 to 62. Plasticity indices for the surficial soils similarly ranged from 18 to 33 while PI at greater depths ranged from 27 to 45. Plastic limits for all soils varied from 10 to 21 and averaged approximately 16. Surficial samples almost universally classified as CL soils while deeper samples classified as either CL or CH soils.

A total of 11 consolidated-undrained triaxial compression tests were performed on specimens from the US54-Fulton site. Stress paths for each of these tests are plotted in Figure 8.3 for specimens from depths less than and greater than 6-ft (1.8-m), respectively. Based on these tests, three different possible failure envelopes were established for the surficial soils and one failure envelope was established for the deeper soils. Mohr-Coulomb effective stress strength parameters for each of these envelopes are summarized in Table 8.1. These indicate that the deeper soils have a significant effective stress cohesion intercept and an effective stress angle internal friction of 25 degrees. The surficial soils have a much smaller cohesion intercept and an angle of internal friction between 23 and 30 degrees.

Table 8.1 Summary of Mohr-Coulomb effective stress strength parameters from triaxial compression tests on specimens from the US54-Fulton test site.

			upper bound A		upper bound B		lower bound	
Stratum	Depths	Sample Numbers	$\bar{c}$ (psf)	$\bar{\phi}$ (°)	$\bar{c}$ (psf)	$\bar{\phi}$ (°)	$\bar{c}$ (psf)	$\bar{\phi}$ (°)
Surficial clay	< 6.0-ft	402, 405 407, 425 147, 148	0	30	91	25	0	23
Deeper clay	> 6.0-ft	185	230	25	--	--	--	--

8.2. Design of Stabilization Schemes

The stabilization schemes selected for the US54-Fulton site were selected based on a series of stability analyses performed for several plausible sets of slope conditions as was done for the previous test sites. The plausible stability cases were again established based on back-analyses performed for the unreinforced slope. For these analyses, the slope was assumed to be homogenous with strength parameters within the range indicated by the failure envelopes described above. Six different pore water pressure conditions were considered. One pore pressure condition assumed negligible pore pressures throughout the slope (i.e. u=0). Another considered a perched water condition within the upper 4-ft (1.2-m) of the slope as was done for previous sites. The remaining pore pressure conditions were defined by piezometric lines passing from the toe of the slope through different levels of the slope ranging from one-quarter of the height of the slope to the full height of the slope. Based on these analyses, the plausible stability cases summarized in Table 8.2 were established. It is noteworthy that each of the plausible stability cases involve a piezometric surface within the slope.

An extensive series of analyses was then performed to evaluate factors of safety for different reinforcement configurations. The reinforcement configurations considered include uniform arrays installed over the entire slide area with member spacings ranging from 3.0- to 6.0-ft (0.9- to 1.8-m). Several additional configurations with non-uniform arrays were also analyzed. A summary of the factors of safety determined for each of these configurations is provided in Table 8.3. Factors of safety for these configurations range from 1.0 to 1.3.

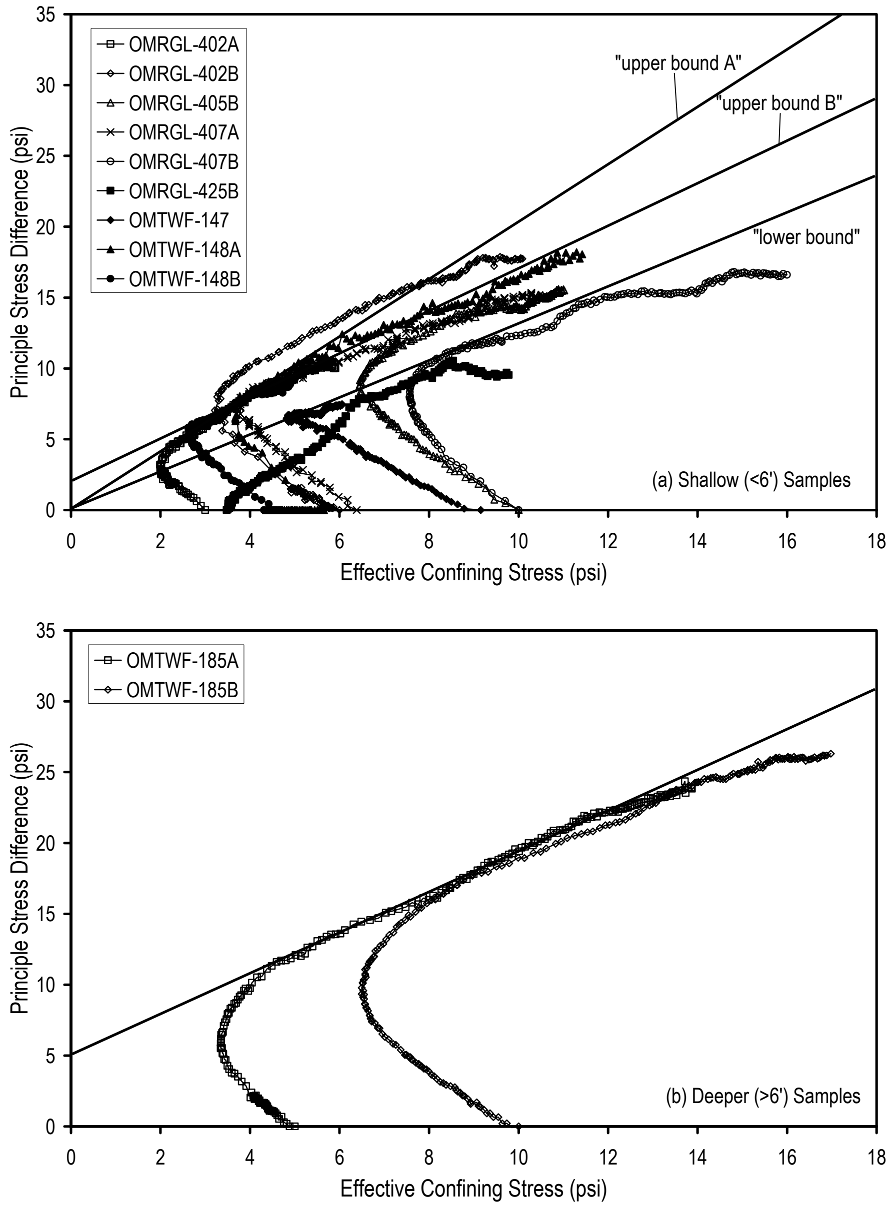


Figure 8.3 Summary of triaxial test results for specimens from the US54-Fulton test site: (a) shallow samples and (b) deeper samples.

The reinforcement configurations selected for use at the US54-Fulton site are shown in Figure 8.4. Several different configurations were again selected to evaluate the effectiveness of alternative stabilization schemes. The slide area was divided into five 40-ft (12.2-m) wide sections, denoted Sections A through E, with a different stabilization scheme selected for each slope section. A 10-ft (3.0-m) wide "separation" between the different reinforced areas was established to address potential concerns about interaction between adjacent test sections. The selected schemes included members placed in uniform grids with member spacings ranging from 4.5- to 10-ft (1.4- to 3.0-m). Greater member spacings were employed at the US54-Fulton site than at previous sites in an attempt to induce failure in one or more sections to allow for rigorous calibration of the design method. Two non-uniform sections were also utilized to evaluate the potential for using more refined grids in critical areas of the slope with sparser configurations in secondary areas. A summary of the estimated factors of safety for each of the selected configurations is provided in Table 8.4. Estimated factors of safety range from approximately 1.15 for Section A with the most refined reinforcement to essentially 1.0 for Sections D and E with the sparsest reinforcement.

Table 8.2 Summary of plausible stability cases leading to the failure at the US54-Fulton test site.

Stability Case	Piezometric line height[1]	Piezometric line distance[2] (ft)	$\bar{c}$ (psf)	$\bar{\phi}$ (°)
A	mid-height	85	0	28.8
B	quarter-height	33 (on face)	51.8	20.3
C	mid-height	87	51.8	20.3
D	mid-height	110	0 51.8	22.4[3] 20.3

[1] piezometric line assumed to vary linearly from toe to height noted at distance noted from toe, beyond which it extends horizontally
[2] horizontal distance from toe of slope to point where piezometric line reaches piezometric line height
[3] stability case D employs bi-linear failure envelope using first envelope for confining stresses less than 8.4-psi and the second envelope at greater confining stresses.

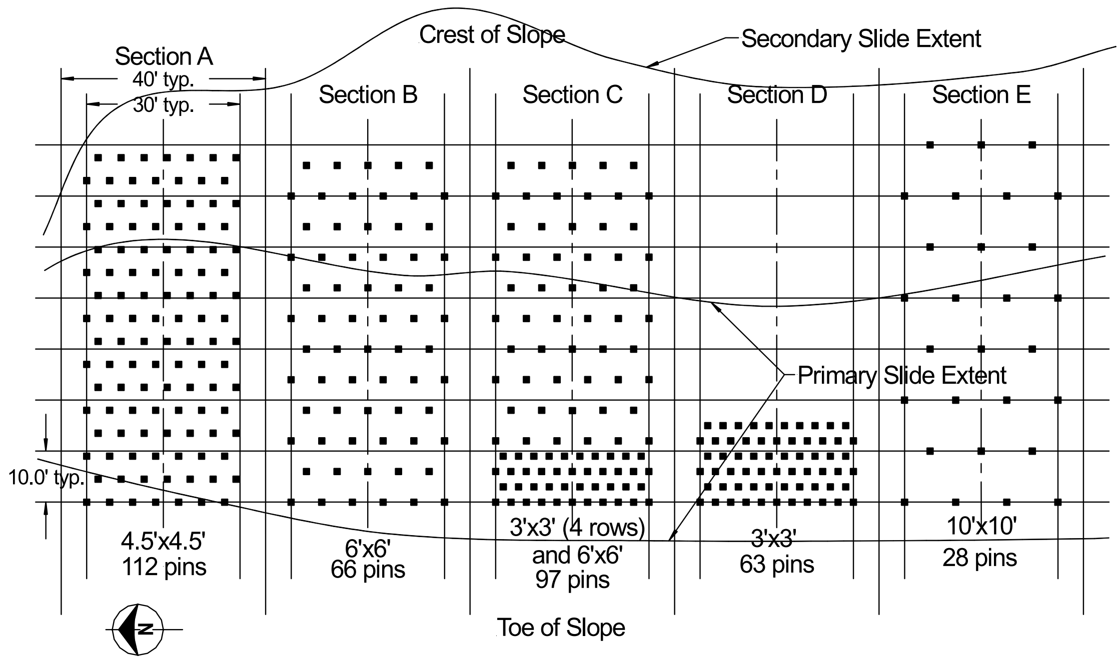


Figure 8.4 Plan view of US54-Fulton site showing selected reinforcement configurations for Sections A through E.

Table 8.3 Summary of factors of safety determined for different reinforcement configurations and stability cases for the US54-Fulton test site.

Rein. Spacing (ft)	Factor of Safety for Respective Stability Case			
	A	B	C	D
3.0L x 3.0T [1]	1.21	1.26	1.16	1.31
4.5L x 3.0T	1.14	1.19	1.10	1.24
6.0L x 3.0T	1.06	1.09	1.07	1.16
3.0L x 6.0T	1.14	1.10	1.07	1.16
4.5L x 6.0T	1.07	1.07	1.04	1.11
6.0L x 6.0T	1.03	1.05	1.03	1.05
3L x 3T on bottom half	1.15	1.24	1.07	1.25
3L x 3T on bottom quarter	1.03	1.09	1.03	1.08
3L x 3T first three rows only	1.00	1.00	1.00	1.02
3L x 3T and 6L x 6T [2]	1.06	1.06	1.03	1.07

[1] L and T denote spacing in longitudinal (strike) and transverse (dip) directions, respectively
[2] 3L x 3T grid on lower three rows, 6L x 6T grid elsewhere

Table 8.4 Summary of estimated factors of safety for each reinforced slope section at the US54-Fulton test site.

Slope Section	Reinforcing Scheme	Estimated Factor of Safety
A	4.5L x 4.5T	1.07 – 1.17
B	6.0L x 6.0T	1.03 – 1.05
C	3.0L x 3.0T (4 rows) 6.0L x 6.0T (rest)	1.03 – 1.07
D	3.0L x 3.0T (6 rows)	1.00 – 1.02
E	10.0L x 10.0T	1.01

8.3. Field Installation

The US54-Fulton site was regraded to the original slope configuration in December 2002. Field installation of the reinforcing member began January 10, 2003 and was completed on January 15, 2003. The equipment utilized for installation was the Ingersoll Rand CM350 that was utilized at the I70-Emma site during Phase II (Figure 7.12). This equipment was able to maneuver on the relatively flat (3.2H:1V) slope without the need for a tether.

A total of 373 recycled plastic members from Batch A10 were installed in the slope over four days. Three additional 3-inch (7.6-cm) diameter landscaping timbers were also installed to evaluate the drivability of timber members. In general, members installed near the top of the reinforcement pattern and members installed near the toe of the slope reached refusal at depths ranging from 3- to 7-ft (0.9- to 2.1-m), while members installed in the middle portion of the slope did not meet refusal. No significant differences were observed in the drivability of the recycled plastic and timber members.

Figure 8.5 shows a photograph of the site near the end of installation. Aside from a minor mechanical problem that delayed installation for one afternoon, the rig was able to install members at a significantly higher pace than was possible at previous sites. The peak installation rate achieved at the site was 141 members/day and all members were installed in just over three working days. Figure 8.6 shows a frequency diagram of the average penetration rates observed for members driven at the US54-Fulton site. Average penetration rates ranged from 0.5-ft/min (0.15-m/min) to over 20-ft/min (6.1-m/min), with a mean rate of 6.6-ft/min (2.0-m/min). While the penetration rates observed at the I70-Emma site and US54-Fulton site during Phase II are somewhat higher than those observed at previous sites, the high installation rates observed at these sites are believed to be primarily due to decreases in "set-up" time between member installations as a result of the ease of maneuvering on the slope and the extendable boom, although the increased penetration rates also contributed to the rapid installation.

Figure 8.5 Photograph of US54-Fulton test site near the end of installation.

8.4. Instrumentation

Instrumentation utilized at the US54-Fulton test site again consists of inclinometers, instrumented reinforcing members, standpipe piezometers, and an array of moisture sensors. Figure 8.7 shows a plan view of the site with the locations of all instrumentation indicated. Six instrumented reinforcing members similar to those used at previous sites were installed during installation. Two instrumented members (IM-20 and IM-25) were placed in Section B while one instrumented member was placed in each of the remaining sections. Member IM-16 was placed in Section A, IM-8 in Section C, IM-21 in Section D, and IM-10 in Section E.

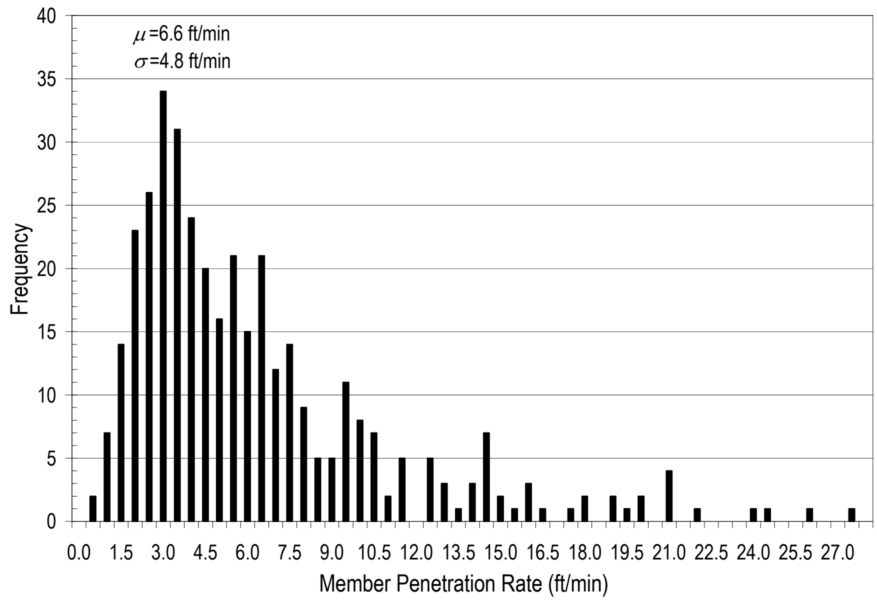


Figure 8.6 Frequency distribution of average penetration rates for recycled plastic members from Batch A10 installed at the US54-Fulton test site. (μ=mean, σ=std. dev.)

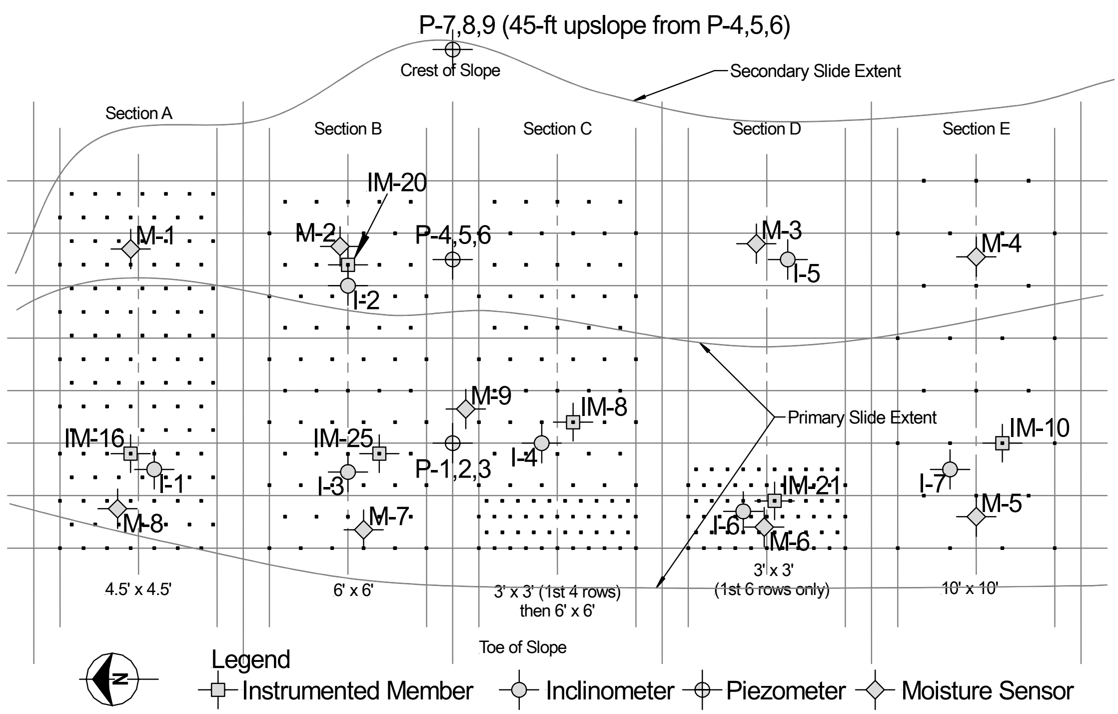


Figure 8.7 Plan view of US54-Fulton test site showing locations of instrumentation.

Seven inclinometer casings were at the site on January 29, 2003. All inclinometer casings were installed within 6-in (15-cm) diameter boreholes extending at least 5-ft (1.5-m) beneath the elevation of the toe of the slope. Six of the inclinometer casings were installed in close proximity to the instrumented reinforcing members; one additional casing was installed in the upper portion of Section D to monitor deformations within the unstabilized area of that section.

Three clusters of standpipe piezometers were also installed near the center of the slide area on January 29, 2003. Each cluster contains three piezometers screened at different depths. Piezometers P-1, P-2, and P-3 were installed in a cluster in the lower portion of the slope and screened at depths of 12.5-, 7.5-, and 2.5-ft (3.8-, 2.3-, and 0.8-m) below grade, respectively. Piezometers P-4, P-5, and P-6 were installed near the upper extent of the stabilized area at depths of 17.5-, 7.5-, and 2.5-ft (5.3-, 2.3-, and 0.8-m), respectively. Piezometers P-7, P-8, and P-9 were installed near the upper extent of the stabilized area at depths of 18.5-, 10.5-, and 2.5-ft (5.6-, 3.2-, and 0.8-m), respectively.

In addition to the standpipe piezometers, an array of moisture sensors was installed at the site on June 6, 2003. One array of two ThetaProbes® and two Equitensiometers® connected to a data logger at location M-9 for essentially continuous monitoring of moisture conditions within the slope. Profile Probe® access tubes were also installed at locations M-1 through M-9 to monitor the vertical and lateral variability of moisture conditions at discrete intervals.

8.5. Field Performance

Field instrumentation at the US54-Fulton site has been monitored since installation was completed in January 2003. The following sections summarize the results obtained from these monitoring activities.

8.5.1. Precipitation at the US54-Fulton Test Site

Figure 8.8 shows the daily and monthly precipitation totals measured at the Columbia Regional Airport located approximately 16 miles (26-km) west of the US54-Fulton test site. While precipitation has increased since installation was completed as is normal for the area, overall precipitation since installation has been well below normal. To date, only two precipitation events greater than 2-in/day (5-cm/day) have occurred at the site. The first occurred in early May 2003; the second occurred in mid-June 2003. Rainfall since the end of June has been very limited.

8.5.2. Piezometers and Moisture Sensors

Water levels measured in each of the nine piezometers installed at the US54-Fulton site are plotted in Figure 8.9. Piezometers P-1, P-6, P-7, P-8, and P-9 have been dry since they were installed in late January. The remaining piezometers have had decreasing water levels, despite the fact that precipitation at the site increased gradually between January and June 2003. Piezometers P-7, P-8, and P-9 are all located high on the slope above the former slide area. The fact that these piezometers have been dry is therefore somewhat expected. Piezometer P-6 is screened at a depth of 3.5-ft (1.1-m) and is located near the upper extent of the former slide area so it is not surprising that this piezometer has also been dry, especially given the lack of significant precipitation at the site. The fact that piezometer P-1 has been dry is more surprising since it is located near the toe of the slope and screened at a depth of

13.5-ft (4.1-m). It is possible that a perched water condition exists above the level where piezometer P-1 is screened. However, additional monitoring is needed to confirm or refute this observation.

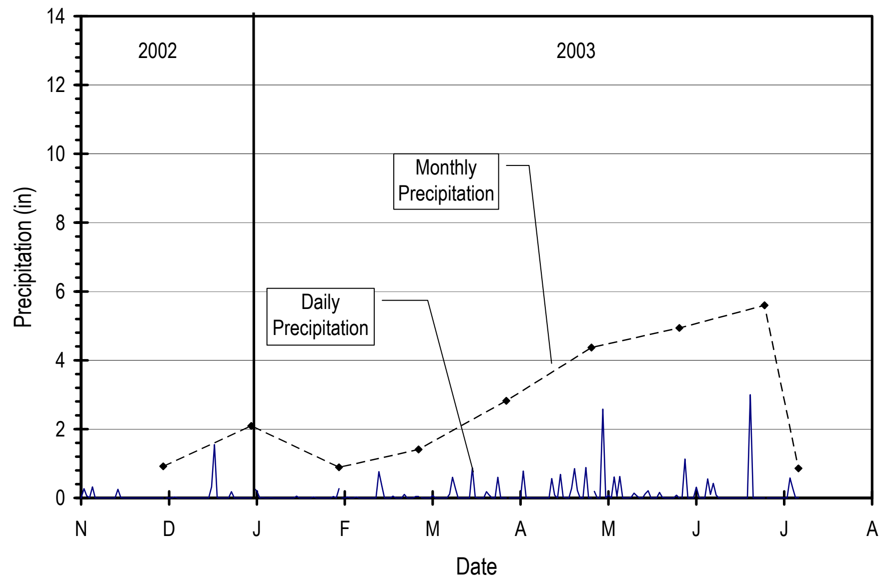


Figure 8.8 Daily and monthly precipitation measured at Columbia Regional Airport approximately 16 miles west of the US54-Fulton site.

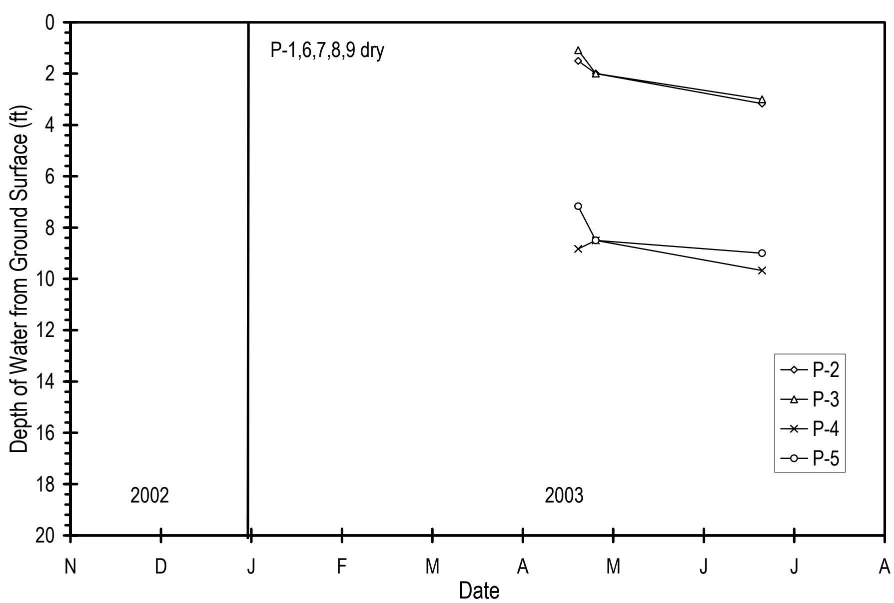
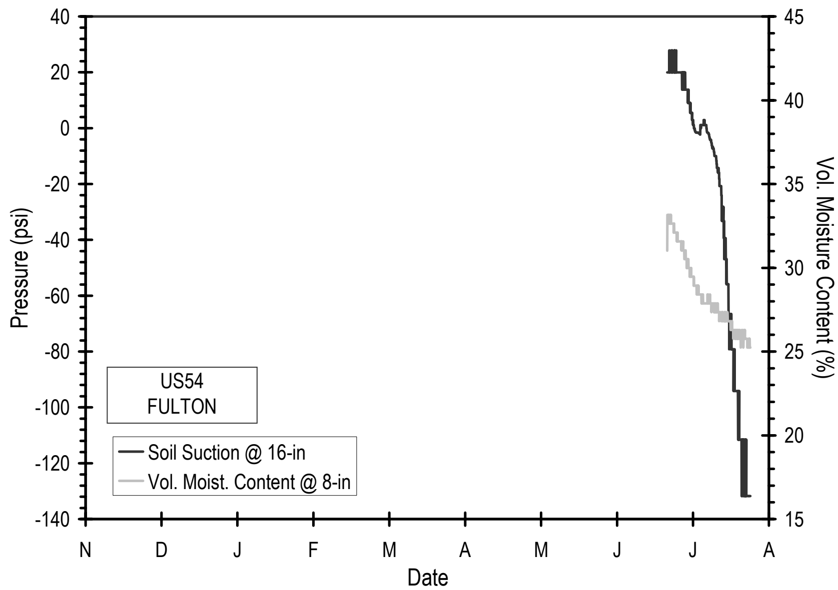
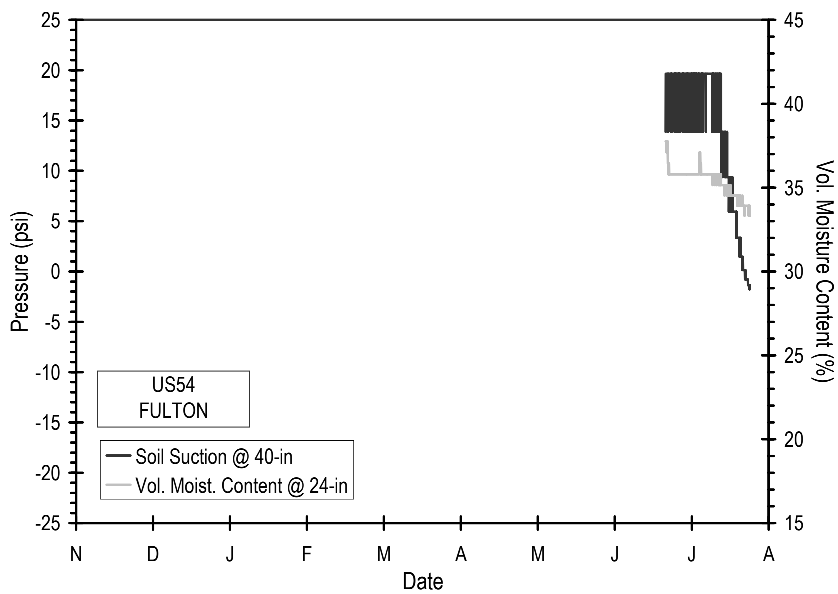


Figure 8.9 Piezometric water levels measured at US54-Fulton test site.

The remaining piezometers seem to indicate that a phreatic condition exists in the lower portions of the slope. Piezometers P-2 and P-3, which are located near the toe of the slope and screened at depths of 8.5-ft (2.6-m) and 3.5-ft (1.1-m), respectively, both indicate similar water levels throughout the monitoring period at relatively shallow depth. Piezometers P-4 and P-5 similarly indicated almost identical water levels, but at greater depth near the upper portion of the former slide area. These data are consistent with a phreatic condition within the slope. Future monitoring of these piezometers when precipitation increases at the site will determine whether such water conditions are representative of those that may have led to the failure of the slope.



(a) shallow sensors



(b) deeper sensors

Figure 8.10 Soil suction and volumetric water content from the US54-Fulton test site: (a) shallow sensors and (b) deeper sensors.

Data from the soil moisture and soil suction sensors installed near the center of the slide area at location M-9 are plotted in Figure 8.10. While no data is available prior to mid-June 2003, the deep sensors both indicated saturated conditions at depths approaching 3-ft (0.9-m) throughout the month of June 2003. This data is consistent with the data from piezometers P-2 and P-3, which both indicate near saturated conditions at shallow depths. During the same time, the shallower sensors indicate decreasing pore pressures and moisture contents, which is indicative of a drying period. Since the beginning of July 2003, all sensors have indicated that pore pressures and moisture contents have decreased significantly, with the shallower sensors indicated lower moisture contents and pore pressures than the deeper sensors as would be expected during a dry period.

8.5.3. Slope Inclinometers

Profiles of the lateral deflections determined from inclinometers I-2 and I-3 are shown in Figures 8.11 and 8.12, respectively For inclinometer I-2, the deflection profile is one in which the maximum deflection occurs near the ground surface and deflections continuously decrease with depth. Inclinometers I-4, I-5, and I-6 have shown similar profiles. The remaining inclinometers (I-1, I-3, and I-7) have deflection profiles similar to that shown for inclinometer I-3 in Figure 8.12. Each of these inclinometers indicate a noticeable discontinuity in the deflection profile at depths greater than 8-ft (2.4-m). While such profiles may be an early indication of sliding, the overall displacements remain relatively small so additional monitoring is needed to determine if sliding is in fact occurring. It is noteworthy to point out that each of the inclinometers showing a discontinuity in the deflection profile is located near the toe of the slope.

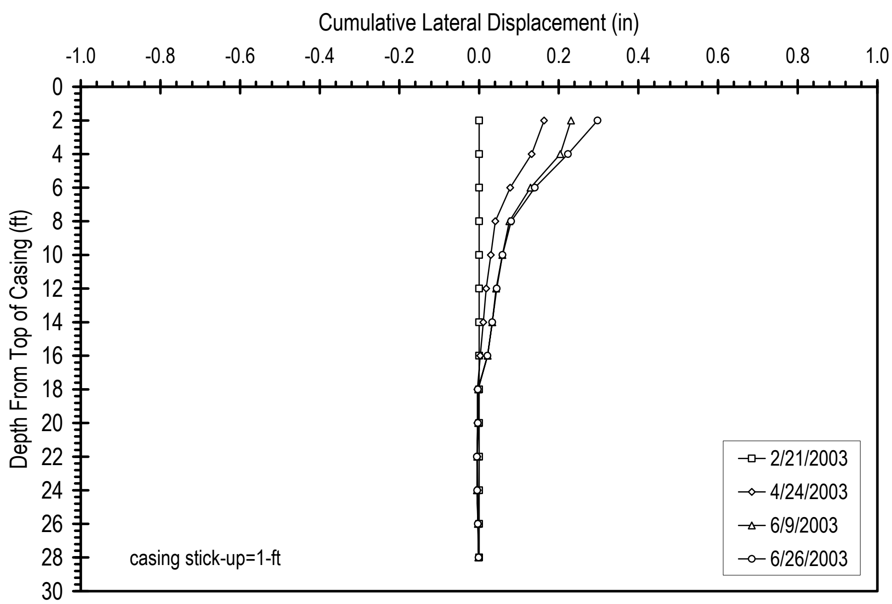


Figure 8.11 Lateral deflection profile for inclinometer I-2 at US54-Fulton test site.

Figure 8.13 shows the maximum cumulative displacements measured in each of the inclinometers plotted as a function of time. All of the inclinometers except inclinometer I-3 indicate gradually increasing displacements with time with a maximum displacement of approximately 0.2-in (0.5-cm) or less. Inclinometer I-3, which is located near the toe of the slope in Section B, has experienced noticeably larger displacements that are approaching 0.8-in (2.0-cm). However, the latest reading indicates that the deformations may be slowing so additional monitoring is needed to establish whether sliding is continuing to occur. It should be noted that inclinometer I-3 is located in Section B, which has members placed on a 6-ft by 6-ft (1.8-m by 1.8-m) staggered grid. The factor of safety estimated for this section is quite low so failure is not out of the question, and in fact would be welcomed as it would allow the design method described in Chapter 3 to be calibrated.

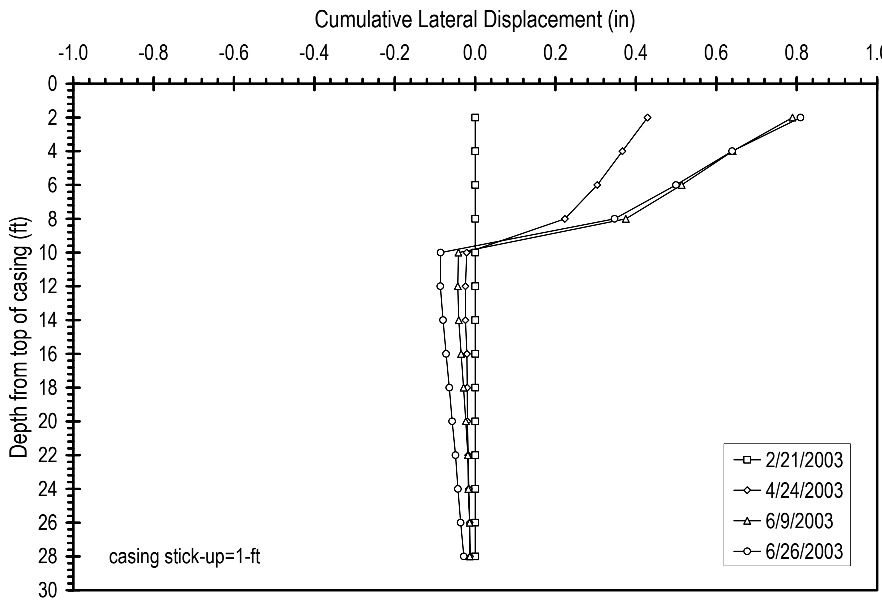


Figure 8.12 Lateral deflection profile for inclinometer I-3 at US54-Fulton test site.

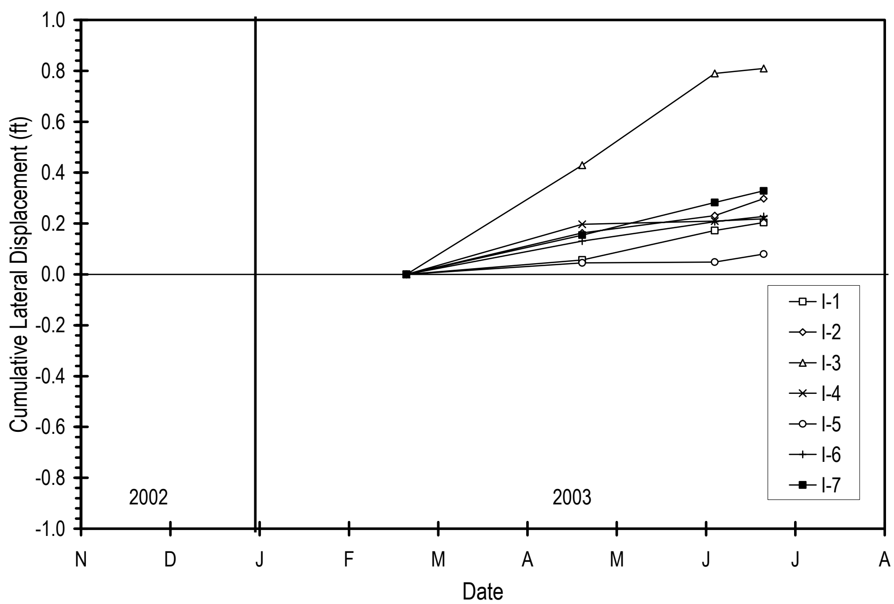


Figure 8.13 Cumulative lateral deflections at depth of 1-ft for inclinometers I-1 through I-7 at the US54-Fulton test site.

8.5.4. Instrumented Reinforcing Members

Four sets of readings have been taken on the instrumented members since installation and these data have been reduced and interpreted to establish the magnitudes of the incremental axial stresses and bending moments in the members. Initial readings prior to installation were only taken on two of the instrumented members at the US54-Fulton site so only incremental stresses and bending moments are presented here.

Figure 8.14 shows the distribution of incremental axial stresses determined for instrumented member IM-25, which is located in Section B near inclinometer I-3. The distribution of axial stresses takes a parabolic form similar to many of the instrumented members at other sites and indicates tensile incremental stresses since installation. Instrumented members IM-8, IM-21, and IM-10 exhibited similarly shaped distributions but with different magnitudes. Instrumented members IM-16 and IM-20 exhibited incremental axial stress distributions with the axial stresses increasing with depth along the member rather than a parabolic distribution with negligible stresses at the two ends of the member.

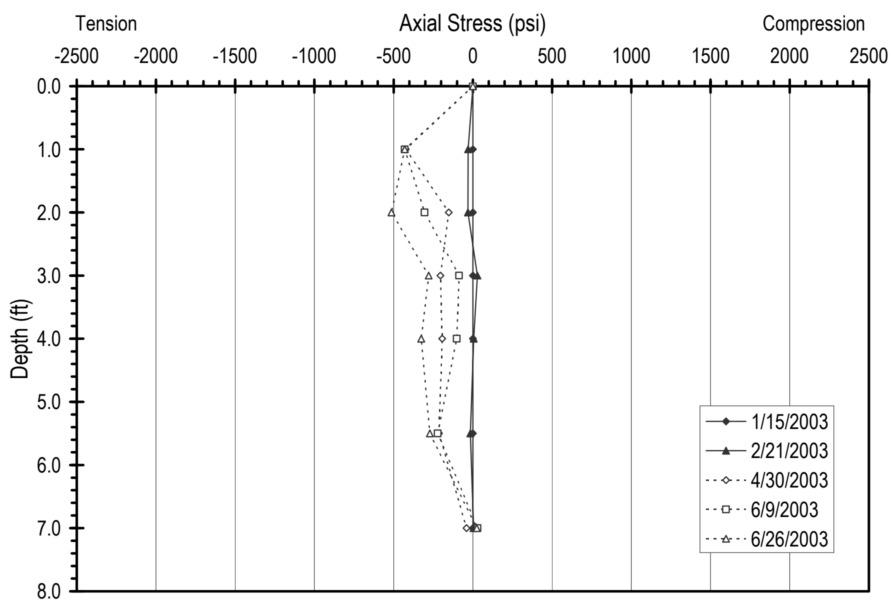


Figure 8.14 Measured incremental axial stresses for instrumented member IM-25 at the US54-Fulton test site.

Figure 8.15 shows the maximum incremental axial stresses for each member plotted versus time. As has been observed for instrumented members at other sites, each of the members has experienced gradually increasing tensile stresses with time. The maximum incremental stresses for all members remain approximately 500-psi (3500-kPa) or less.

Two different forms of bending moment distributions have been observed for the instrumented members at the US54-Fulton site as shown in Figures 8.16 and 8.17 for members IM-25 and IM-8, respectively. Members IM-25, IM-10, IM-16, and IM-20 all showed S-shaped distributions of bending moments with positive bending moments in the upper portion of the member and negative bending moments in the lower portion (Fig. 8.16). Members IM-8 and IM-21 had bending moment distributions with positive moments along the entire length of the member with the maximum bending moment occurring at a depth of

2- to 3-ft (0.6- to 0.9-m). Both forms of bending moments have been observed at previous sites.

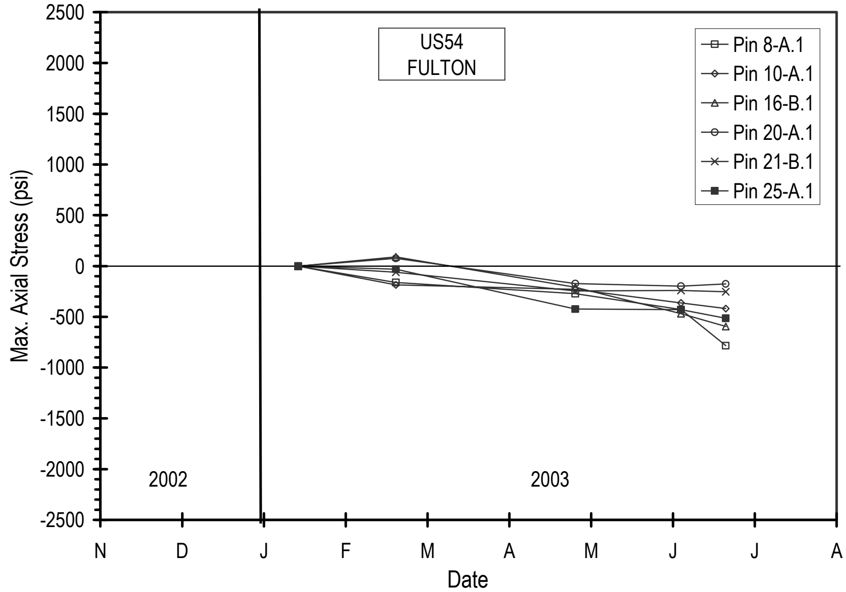


Figure 8.15 Maximum incremental axial stresses in instrumented members at US54-Fulton test site.

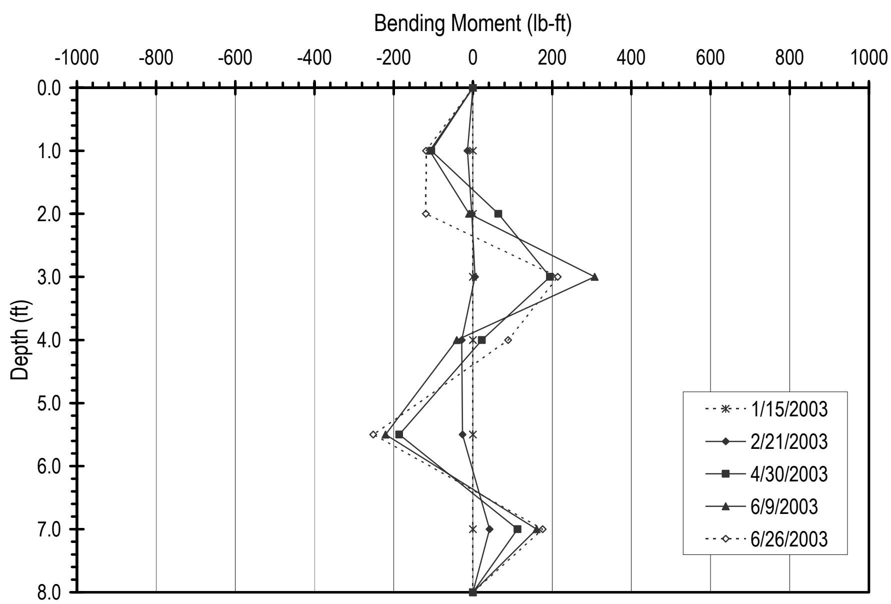


Figure 8.16 Measured incremental bending moments for instrumented member IM-25 at the US54-Fulton test site.

Figure 8.18 shows the maximum bending moments measured in each of the instrumented reinforcing members from the US54-Fulton site plotted as a function of time. To date, the incremental bending moments measured for each of the members have increased with time but remain relatively small. The maximum bending moments for members IM-10, IM-16, IM-20, and IM-21 have increased in an approximately linear manner. All of these members have maximum bending moments that are less than 200-lb-ft (270-N-m). In contrast, maximum bending moments for members IM-8 and IM-25 increased at an increasing rate between February and June 2003 and both of these members have experienced maximum incremental bending moments of approximately 300-lb-ft (400-N-m), which is significantly higher than that determined for the other members at the site. No clear reason is apparent for the higher bending moments for these two members; however, they are both located in close proximity to one another in the lower portion of the slide in Sections B and C.

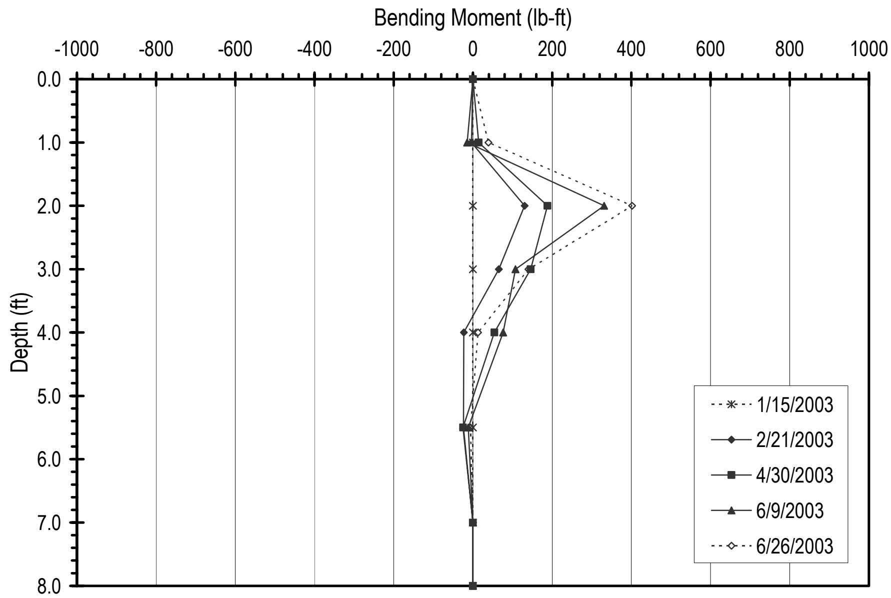


Figure 8.17 Measured incremental bending moments for instrumented member IM-8 at the US54-Fulton test site.

Overall, the instrumentation at the US54-Fulton site indicates that, despite having lower than normal precipitation at the site since installation, the reinforcing members have begun to carry some load associated with soil movements. Instrumented members IM-8 and IM-25 appear to be carrying the largest loads at this time. The fact that member IM-25 is carrying significant loads is consistent with the larger movements observed for inclinometer I-3, which is located near to member IM-25. It is also consistent with the fact that the stabilization pattern in Section B, where member IM-25 is located, is one of the most widely spaced patterns used at the site. Lateral deformations for inclinometer I-4, which is located near member IM-8, are not as large as those observed for inclinometer I-3. However, IM-8 is located in an area of Section C where reinforcing members are placed in a pattern similar to that used in Section B. Other reinforcing members have also begun to carry some load. However, the loads are significantly less than those for members IM-8 and IM-25, which is consistent with the relatively small displacements that have been observed in inclinometers near the respective members.

It is also interesting to note that both the incremental axial stresses and the incremental bending moments observed at the US54-Fulton test site are generally larger than those that have been observed for slide area S3 at the I70-Emma site. Several possible reasons exist for this observation. One possible explanation is simply that the slopes have been subjected to different conditions since installation. However, the stabilized sections also differ in that the I70-Emma slope is much steeper than the US54-Fulton slope. The I70-Emma slope is also an embankment slope, while the US54-Fulton slope is an excavated slope. Whether either of these facts has a significant influence on the observed behavior is not yet known. Finally, it should be noted that the reinforcing members at the I70-Emma site were installed with a vertical orientation while the reinforcing members at the US54-Fulton site were installed perpendicular to the face of the slope. The load transfer model developed based on observations at previous sites would suggest that higher incremental axial stresses and bending moments would be expected for members installed perpendicular to the face of the slope. The data acquired from the US54-Fulton site provides further support for this conceptual model. It is hoped that continued monitoring of these sites as well as additional analysis and interpretation of the field data will help to further confirm or refute these observations.

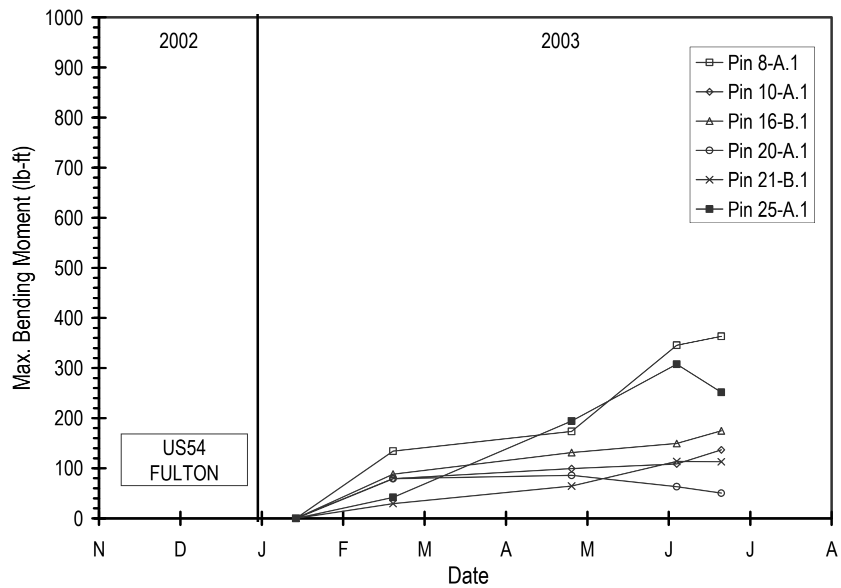


Figure 8.18 Maximum incremental bending moment in instrumented members at US54-Fulton test site.

8.6. Condition of Instrumentation at US54-Fulton Test Site

All of the instrumentation installed at the US54-Fulton site appears to be performing well. Additional monitoring is therefore expected to produce valuable data to evaluate the performance of the five stabilized sections and establish the load transfer mechanisms for the reinforcing members in each section.

8.7. Summary

Activities undertaken to establish the US54-Fulton test site have been described in this chapter. The slope at the site is a relatively flat, but long excavated slope in ablation till. Five different stabilized sections with different reinforcement configurations were constructed to evaluate the effectiveness of different stabilization schemes. Installation of the reinforcing members went extremely well with almost 400 members being installed in less than four working days. The increased pace of installation was a result of the increasing experience of the installation personnel and improved installation equipment that was well suited to the installation. To date, results obtained from monitoring of field instrumentation at the site indicate that all of the stabilized sections appear to remain stable. However, precipitation at the site since installation has been below normal levels so additional monitoring is needed to truly evaluate the effectiveness of the different schemes.

Chapter 9. Implications of Field Performance to Date

As of July 2003, six different surficial slides at four different sites have been stabilized using recycled plastic reinforcing members. One additional slide has been stabilized using similarly sized steel pipe. The performance of each of these sites has been monitored since each slope was stabilized. The periods of monitoring for the sites ranges from a minimum of six months to over three years. While additional monitoring is needed at several sites, the performance of each site to date is sufficient to begin to draw a number of conclusions regarding different aspects of the research project. The following sections present these preliminary conclusions with respect to the overall effectiveness of the stabilization method, the mechanism(s) through which load is transferred to the reinforcing members, the suitability of the design method established during Phase I, the constructability of the method, and the cost-effectiveness of the stabilization method. Several lessons learned regarding instrumentation of the field test sites are also presented along with discussion of future implementation of the slope stabilization technique on a more widespread basis.

9.1. Overall Effectiveness of Stabilization Scheme

The potential effectiveness of using relatively slender recycled plastic reinforcing members has been demonstrated through successful stabilization of six different surficial slides at four different sites for periods ranging from six months to over three years. To date, all stabilized sections remain stable. Control sections at the I70-Emma site during Phase I have failed while controls sections established during Phase II have yet to fail.

These observations indicate that slopes can be effectively reinforced to prevent surficial slope failures using recycled plastic reinforcing members. However, crucial information regarding how closely the reinforcing members must be placed to provide effective long-term stabilization is still lacking. The performance of the slopes at the I70-Emma site during Phase I and the I435-Wornall Road site during Phase II provides strong evidence that placing reinforcing members in a 3-ft by 3-ft (0.9-m by 0.9-m) staggered grid pattern across the entire slide area is sufficient for long-term stabilization. The stability of test sections established with more widely spaced reinforcing members also suggests that the slopes may possibly be stabilized using more widely spaced reinforcement configurations that are more economical. However, some caution regarding the effectiveness of more widely spaced reinforcing members is warranted since the effectiveness of such schemes has not truly been demonstrated because of a lack of normal to above normal precipitation at the sites. Data from the instrumentation at each site clearly indicates that, while some movement has occurred, the test sections have yet to mobilize significant resistance in the reinforcing members and have yet to reach an equilibrium condition similar to what has been observed at the I435-Wornall Road and I70-Emma test sections. Additional monitoring of the performance of these sites is therefore needed to more definitively establish the effectiveness of these more economical stabilization schemes.

9.2. Load Transfer Mechanisms

In evaluating the field instrumentation data for all sites collectively, a consistent pattern of behavior has been observed. This pattern generally consists of the following three stages of behavior:

(1) A period of several months following stabilization where limited movements are observed and little load is mobilized in the reinforcing members aside from initial stresses developed during installation.

(2) A period of several months during which slope movements are observed to increase significantly while the loads in the reinforcing members are simultaneously observed to increase; and

(3) An extended period where slope movements are observed to diminish at the same time as the loads in the reinforcing members become essentially constant.

All three of these stages of behavior have been observed at both the I70-Emma site during Phase I and the I435-Wornall Road slope during Phase II. The behavior of the slopes at the remaining sites indicates that these slopes are somewhere within Stages 1 or 2.

It is postulated that the observed behavioral pattern for the stabilized slopes is a combined result of the pore pressure conditions within the slope and mobilization of resistance in the reinforcing members. Just after installation, the slope would be stable without the reinforcing members because of the low pore water pressures within the slope. However, during the first period of extended heavy precipitation following installation, the stability of the slope decreases in response to increasing pore water pressures. As the stability decreases, the slope begins to move at which point the reinforcing members begin to deflect and mobilize some resisting loads. With continued higher pore pressures, slope movements continue until the reinforcing members mobilize sufficient resistance to create equilibrium in the slope. At this point, additional movement is resisted by the reinforcing members and movement essentially ceases. Upon subsequent wetting and drying, the resistance in the members is already mobilized which prevents significant additional movement unless the pore water pressures become significantly greater than have been experienced in the slope since installation. In cases where subsequent pore pressures are greater than previously experienced, additional movement would begin to mobilize additional resistance until a new equilibrium condition is reached as long as no limit state is reached to produce failure. Continued monitoring of the field tests sites will be used to confirm or refute this postulated behavioral pattern for slopes with different stabilization schemes and different conditions.

Several additional observations regarding the load transfer mechanisms can also be made based on results obtained from the instrumentation data obtained to date. One clear observation that can be made is that measurable axial stresses and bending moments may be induced in the reinforcing members during installation. However, the potential effects of these initial stresses are not immediately clear. On one hand, the initial stresses are expected to affect how much additional load the reinforcing members can take prior to failing. However, whether this effect is positive or negative is not obvious. This issue can be illustrated by considering a hypothetical member with significant initial compressive stresses developed during installation. If the member is subsequently subjected to additional compressive stresses imposed due to slope movements, one would expect that the member might fail at conditions less than those assumed in design (where initial stresses are neglected). In such an instance, the "available capacity" of the reinforcing member is reduced below that considered by assuming the member to be unstressed following installation. In contrast, if the stresses imposed due to slope movements were of opposite sign (i.e. tensile), the stresses imposed due to slope movements would first have to overcome the initial compressive stresses prior to imposing actual tensile stresses on the members (similar to prestressing/post-tensioning of concrete). In this case, the "available capacity" of the member is increased above that considered by assuming the member is unstressed following installation and the member might not reach failure until conditions were much worse than assumed in the design. A similar example can be made for bending moments or any other loads imposed on the members. It is therefore relatively clear that the initial stresses may affect the magnitude of the additional stresses or bending moments that can be imposed on the members due to movement of the soil, but it is not clear what this effect might be or how these effects should be considered in design.

It is also not clear whether the initial stresses or bending moments imposed in the members during installation will contribute to the stability of the slope. For example, if a member was installed such that the member had significant positive bending moments, one might expect that significant lateral stresses might be transferred to the soil to resist slope movements. Similarly, if a member was installed so that significant negative bending moments were developed, one might expect that lateral stresses that tended to promote slope movements could exist. Additional analyses are therefore needed to evaluate the effect of initial stresses and bending moments on the overall stability of the slope. Regardless of the above issues, it is clear that the form of the stresses or moments imposed on the members *due to slope movements* can only be accurately evaluated by considering the stress *changes* imposed since installation because the initial loads can take on a variety of forms that may not be logically associated with slope movements.

Data obtained from the field instrumentation also indicates a difference in the pattern of load transfer for members installed vertically and members installed perpendicular to the face of the slope. Data from sites where reinforcing members were installed vertically (US36-Stewartsville site and slide areas S2 and S3 at the I70-Emma site) generally indicates that members installed vertically experience smaller changes in both the axial stresses and bending moments than sites where the reinforcing members were installed perpendicular to the face of the slope (slide area S1 at the I70-Emma site and the US54-Fulton site) at similar stages of behavior. This would suggest that it is preferable to install members vertically in order to minimize the loads on the reinforcing members. However, it is also important to consider the ramifications of member orientation on the overall stability of the slope, rather than simply on the loads within the reinforcing members. Additional study is needed to evaluate the effect of these apparent load transfer mechanisms on the overall stability of the stabilized slopes and on the relative effectiveness of members installed at different orientations. It should also be noted that data from the I435-Wornall Road site, where reinforcing members were installed vertically, indicates loads that were higher than observed at other sites. However, as discussed in Chapter 5, data from the strain gages at the I435-Wornall Road site is believed to be the least reliable of all of the strain gage data so additional analysis is needed to confirm the results from the I435-Wornall Road site.

9.3. Suitability of Design Method

In general, the performance of the field test sites indicates that the general design approach developed during Phase I is suitable for estimating factors of safety for different reinforcement configurations. However, several possible modifications to the specific design method need to be evaluated to address issues that have arisen from observation of the field test sites.

The first modification to be evaluated is to incorporate consideration of contributions from axial stresses to the overall stability of the reinforced slopes. While the potential existence of axial forces in the reinforcing members has been recognized since the beginning of the project, axial forces have been ignored to date because of lack of information regarding the magnitude, and even the sign, of these forces. However, given that additional information regarding the potential magnitude of the axial forces is now available from field performance monitoring, the design method can be modified to evaluate the effect and importance of these axial forces. In the final analysis, it may still be recommended that axial forces be neglected for general design because of large uncertainties regarding the magnitude of the forces. Nevertheless, it is important that the potential contributions of these forces be given consideration during Phase III so that sound recommendations can be made regarding the influence of member inclination and the issue of whether or not axial forces should be considered in design.

The second modification to be evaluated is the method used for estimating the limiting soil pressure in the design procedure. The performance of slopes at the I70-Emma site during Phase I and the I435-Wornall Road site during Phase II sites indicates that the design method is conservative, and perhaps overly conservative, because slopes with factors of safety just greater than unity have remained stable for several years. One source of this conservatism is the method used for estimating the limiting soil pressure. As described in Chapter 3, the method by Ito and Matsui (1975) has been used throughout the project. One reason for selecting this method is that it is believed to be conservative, and the field data obtained to date seem to confirm this belief. Additional analyses therefore need to be performed to evaluate other methods for predicting the limiting soil pressure with the hopes that they may provide more accurate factors of safety. This process is complicated by the fact that there is no generally accepted relation between values of the factor of safety and observed slope movements. However, additional stability analyses for as built conditions and field measured water conditions, coupled with more advanced stress-deformation analyses, will allow such a relation to be developed for each of the field test sites. This relation can then be used to develop appropriate levels of the factor of safety for acceptable performance (based on deformations or other factors). This process will be greatly facilitated if a failure should occur in one or more of the field test sections because such failures will provide a "ground truth" condition that can be used to calibrate the design method more accurately.

9.4. Constructability

Efforts at the I70-Emma test site during Phase I of the project demonstrated that recycled plastic members could be successfully installed in a slope with enough efficiency to serve as a cost effective alternative for stabilizing surficial slope failures. Installation efforts during Phase II confirmed this conclusion and, further, demonstrated that installation rates could be significantly improved over those achieved during Phase I with better equipment and more experience.

Four different pieces of installation equipment were utilized during Phase II. Photographs of each piece of equipment can be seen in Figures 5.11, 7.10, and 7.12. While the four rigs used for installation differ in several respects, the common characteristic among all of the rigs was that each rig had some form of mast to maintain the alignment between the hammer and the reinforcing member. The importance of maintaining this alignment was clearly demonstrated in Phase I (Loehr et al., 2000).

Table 9.1 shows a summary of information related to installation of the reinforcing members at each of the respective test sites. In the table, the sites are listed from top to bottom in chronological order of the installations. The information provided shows that installation rates – the rate of installation including necessary set up time between members – improved significantly as more experience was gained with the installation technique and minor modifications were made to the installation equipment. Installation rates achieved at the first test site during Phase I reached a maximum of approximately 80-members/day. Installation rates achieved at the final test site to be established (the US54-Fulton site) reached a maximum of 140-members/day, which is almost double that achieved at the I70-Emma test site during Phase I. Similar installation rates appear to be a reasonable expectation for future sites with similar installation equipment although higher rates may be possible in the future if installation equipment specific to this application is developed.

Table 9.1 Summary of installation data for each of the field test sites.

Field Test Site	Working Days	Members Installed	Installation Equipment
I70-Emma, Slide Area S1	1	45[1]	Okada OKB 3051250 ft-lb energy class hydraulic hammer
	2	154	Davey-Kent DK 100B crawler mounted hydraulic drill
I70-Emma, Slide Area S2	3	163	Davey-Kent DK 100B crawler mounted hydraulic drill
I435-Wornall Road	2	33	Davey-Kent DK 100B crawler mounted hydraulic drill
	10	583	Ingersoll Rand CM150, IR 350 CFM, 100 psi air compressor
I435-Holmes Road	5	262[2]	Ingersoll Rand CM150, IR 350 CFM, 100 psi air compressor
US36-Stewartsville	5	360	Ingersoll Rand CM150, IR 350 CFM, 100 psi air compressor
I70-Emma, Slide Area S3	2	166	Ingersoll Rand ECM350, IR 300 CFM, 100 psi air compressor
	1	32[1]	Daken Farm King hitter series II, Case XT90 skid steer loader
US54-Fulton	4	377	Ingersoll Rand ECM350, IR 300 CFM, 100 psi air compressor

[1] denotes trial installation
[2] denotes steel pipe members

Penetration rates – the rate of penetration for an individual member excluding set up time – were also measured during installation at each of the test sites. In general, penetration rates also increased as more experience was gained with the installation procedure and as better equipment was utilized. Penetration rates measured during the installation at the I70-

Emma site during Phase I averaged slightly over 4-ft/min (1.2-m/min) while penetration rates measured at the US54-Fulton test site averaged 6.6-ft/min (2.0-m/min). While the increase in penetration rates certainly contributed to the large increase in installation rates, the high overall installation rates achieved at the most recent sites would not have been possible without simultaneous decreases in the set-up time required between member installations. It is also noteworthy to point out that neither the overall installation rates nor the individual penetration rates appeared to change significantly when members of different types (e.g. steel pipe, timber, other recycled plastic members) were installed on a trial basis (Chen, 2003; Bowders et al., 2003).

9.5. Economic Considerations

During Phase I, the costs to install the reinforcing members in slide areas S1 and S2 was just under $4.00/ft^2 ($45/m^2). Estimated costs to stabilize the same slope using the common method of removal and replacement (with large aggregate) were approximately $5.50/ft^2 ($60/m^2). Costs incurred for stabilization of the test sites established during Phase II were tracked and analyzed to expand the database of costs for the stabilization method. These costs are summarized in Table 9.2.

In general, the cost of the recycled plastic members used at each of the test sites was nominally $20/member. Costs for the similarly sized steel pipe utilized at the I435-Holmes Road site were similar when the costs for backfilling the pipes with lean grout were included. Labor costs varied somewhat, but were nominally $20/member. Cumulative costs for each site are reported in two different ways in Table 9.2. The first method used to calculate cumulative costs for each site was to simply take the average unit cost for materials ($20/member) and installation ($20/member) and multiply that cost by the number of members installed. These costs are reported as "Total Costs" in Table 9.2. The second cumulative cost report for each site is the actual cost paid for each installation, which are reported as "Actual Costs" in Table 9.2. Actual costs for the I70-Emma site stabilization during Phase I and for the I435-Wornall Road and I435-Holmes Road stabilizations during Phase II were paid on a lump sum basis. The actual costs paid therefore differ from the "total" costs estimated. The remaining stabilizations were paid on a unit cost basis. The "total" and "actual" costs are therefore identical. Unit costs reported in the table were computed using the "Total Costs" rather than the actual costs paid to provide a consistent basis for the unit costs at each site.

Overall, unit costs for the different stabilization schemes utilized at the different sites varied substantially. Unit costs for slope areas stabilized using a 3-ft by 3-ft (0.9-m by 0.9-m) staggered arrangement of reinforcing members were approximately $4.50/ft^2 ($50/m^2). In contrast, unit costs for slope areas stabilized using a 6-ft by 6-ft (1.8-m by 1.8-m) staggered arrangement of reinforcing members is only $1.00/ft^2 ($11/m^2). These results clearly indicate the significant economic benefits that can be realized if reinforcement spacing can reliably be increased while still providing a reasonable margin of safety.

9.6. Effectiveness of Instrumentation

Overall, the field instrumentation has proven to be extremely useful for monitoring the performance of the respective stabilization sites and for establishing both a qualitative and quantitative understanding of the behavior of the slopes at these sites. Without this information, many of the conclusions described in this chapter could not have been drawn and little could be said other than that each of the stabilized sections has remained stable. The value of the field instrumentation is therefore readily apparent. However, some problems with some of the instrumentation have been encountered as described below.

Table 9.2 Summary of cost analyses for stabilization of field test sites.

Field Test Site	Slope Section	Stabilized Area WxL (ft)	Member Spacing LxT[1] (ft)	Members Installed	Total Cost[2] ($)	Unit Cost[3] ($/ft^2)	Actual Cost[4] ($)
I70 Emma	S1	42x42	3x3	199	7960	4.5	11590
	S2	39x36	3x3	163	6520	4.6	
I435-Wornall		115x76	3x6; 3x3[5]	916	36640	4.2	44740
I435-Holmes		60x51	3x3	262	10480	3.4	18315
US36-Stewartsville	A	30x66	4.5x3	161	6440	3.3	14600
	B	30x66	6x6	54	2160	1.1	
	C	31x66	6x4.5	67	2680	1.4	
	D	32x66	4.5x6	78	3120	1.6	
I70-Emma	S3-A	25x48	4.5x3	95	3800	3.2	7960
	S3-B	25x36	4.5x6	35	1400	1.6	
	S3-C	25x36	6x6	30	1200	1.3	
	S3-D	25x36	6x4.5	38	1520	1.7	
US54-Fulton	A	34.5x75	4.5x4.5	113	4520	1.7	15040
	B	36x75	6x6	66	2640	1.0	
	C	36x75	3x3; 6x6[5]	97	3880	1.4	
	D	33x75	3x3	73	2920	1.3	
	E	40x75	10x10	28	1120	0.4	

[1] longitudinal (strike) spacing measured parallel to roadway by transverse (dip) spacing measured along slope
[2] total cost based on $20/member material costs and $20/member installation costs
[3] total cost divided by stabilized area
[4] actual costs paid for stabilization as part of the project
[5] mixed reinforcement configuration utilized

Equitensiometers® installed at the I435-Wornall Road site have been out of range since they were installed. While this suggests that the sensors are not providing the information desired, the fact that the sensor readings are out of range at least confirms the belief that positive pore pressures exist within the upper stratum at the site. Such confirmation would not have been possible without the sensors. Nevertheless, in hindsite, it would have been better to install positive pore pressure sensors at the I435-Wornall Road site to obtain quantitative data on the magnitude of the field pore pressures. Consideration should be given to installation of such sensors at the I435-Wornall Road site during Phase III, since it has become clear that positive pore pressures dominate at this site. It should be noted that Equitensiometers® installed at other test sites have provided good quantitative information.

The ThetaProbes® installed at each site have been performing quite well and, to date, they have provided valuable information for developing a qualitative understanding of the conditions within each of the slopes. However, additional work is needed to establish the soil-water characteristic curves for the soils at each site so that the soil moisture data can be correlated with soil suctions so that quantitative estimates of the pore pressures within the slopes can be obtained. Such work is underway and will continue into Phase III to allow the soil moisture readings to be correlated with actual pore pressures that can be used in evaluating the field performance of the test sites.

Strain gages have proven to be effective at establishing the loads mobilized in the reinforcing members, despite the fact that significant assumptions must be made to interpret the data. Data from all of the field test sites to date has indicated a consistent pattern of load transfer as described above. This pattern could not have been identified without the benefit of the strain gage data. However, it is clear at this stage of the project that the strain gages have a limited useful life after which a sufficient number of gages become inoperable to make establishing reliable estimates of the loads in the members extremely difficult. Despite this limitation, no other strain gages with acceptable characteristics could be identified so the choice of gages is believed to have been a sound decision.

Another issue associated with the strain gages has yet to be resolved. This issue is whether or not the strain gages have accurately captured the magnitude of the stresses and bending moments imposed in the members during installation. All indications to date suggest that the gages did not accurately capture these stresses and bending moments. However, additional analyses of the instrumentation data are needed before this conclusion can be confirmed.

Finally, the force sensing resistors (FSR) used on all of the instrumented reinforcing members have produced data that is of limited value. The problem has been that all of the FSR readings have remained below the "detection limit". While this has limited the value of having these sensors installed on the reinforcing members, the readings do serve to provide an upper-bound on the magnitudes of the lateral stresses applied to the members, which has and will continue to be useful information when reducing the data from the strain gages.

9.7. Implementation Issues

At this point in time, many of the issues associated with using recycled plastic reinforcing members to stabilize surficial slope failures have been addressed. Construction method(s) have been established that provide for reliable and efficient installation of the reinforcing members without imposing significant damage to the members. A preliminary specification for recycled plastic reinforcing members has also been developed and is currently under review (Chen, 2003; Bowders et al., 2003). Finally, performance monitoring at several field test sites indicates that installation of reinforcing members on a 3-ft by 3-ft (0.9-m by 0.9-m) staggered grid is likely to be sufficient to stabilize many surficial slope failures. Based on these observations, it seems appropriate that the technique can begin to be implemented at other sites on a trial basis when appropriate sites are identified.

Several additional issues remain to be addressed however prior to widespread implementation. Perhaps the primary remaining issue to be addressed is whether placing reinforcing members in more economical arrangements will also be sufficient to stabilize surficial slides. Field performance data obtained to date for slopes stabilized with more widely spaced reinforcing members has generally been inconclusive. Additional monitoring of the field performance of the test sites established during Phase II is expected to provide data to more definitively address this issue. In the best case, one or more of the stabilized sections will experience a failure, which will provide a definitive case that can be used to calibrate the design methodology. However, even if a failure is not observed, additional monitoring coupled with additional analyses of the results of this monitoring will permit the design method to be improved to more accurately predict the effectiveness of alternative stabilization schemes for different slope conditions. Additional issues that must be addressed prior to widespread implementation include development of simple design tools, such as design charts or rules of thumb, final adoption of a standard specification for recycled plastic reinforcing members, and development of technology transfer documents or activities (e.g. videos, continuing education courses) to help educate personnel on proper design and construction of the stabilization measures.

9.8. Summary

In this chapter, a number of different conclusions have been presented based on the results obtained from this project to date. Results of monitoring of the field test sites established during Phases I and II indicates that surficial slope failures can be stabilized using recycled plastic reinforcement placed in a 3-ft by 3-ft (0.9-m by 0.9-m) staggered grid over the entire slide area. Stabilization may also be possible using more widely spaced reinforcement with significant associated reductions in overall costs. However, additional monitoring of field tests sites where more widely spaced reinforcing members have been utilized is needed to reliably confirm this possibility and to establish the relation between reinforcement spacing and the reliability of the stabilization measures. The performance observed at all test sites to date also suggests that each of the stabilized sections is following a consistent, three stage pattern of behavior with the duration of each stage being dependent on the pore pressure conditions within the slope and the mobilization of resistance in the reinforcing members. Several recommended modifications to the design procedure developed during Phase I were then presented followed by a summary of observations regarding the installation rates achieved at the field test sites. The costs associated with each of the stabilized sections were then summarized and discussed. Finally, several comments regarding the effectiveness of the instrumentation used to monitor the performance of the field sites and further implementation of the slope stabilization technique were presented.

Chapter 10. Summary, Conclusions, and Recommendations

Activities undertaken to further evaluate the use of recycled plastic reinforcing members for stabilization of surficial slope failures during Phase II of the project have been summarized in this report. This chapter includes a brief summary of items included in the body of the report along with a number of broad conclusions drawn from the project activities. Recommendations for future work to address three significant issues before widespread implementation can be realized are also presented.

10.1. Summary

The report is organized in ten chapters, each of which covers a different aspect of the project. Chapter 1 of the report included a brief section describing the motivation behind the project entitled "Slope Stabilization Using Recycled Plastic Pins". Relevant background information summarizing the work performed in Phase I of the project was then presented followed by a summary of the objectives and specific tasks undertaken during Phase II.

The process for selection of the field test sites utilized in Phase II was presented in Chapter 2. The general criteria used to screen candidate sites and to make the final site selections were described. General characteristics of the most promising of these sites were then presented along with a brief justification for selection of the sites established during Phase II.

The design method developed during Phase I, and modified during Phase II, was described in Chapter 3. In this chapter, the general approach adopted for analyzing the stability of reinforced slopes was first described followed by descriptions of the specific methods used to predict the resistance provided by individual recycled plastic reinforcing members considering several potential limit states.

Additional efforts undertaken during Phase II to evaluate the properties of recycled plastic members were described in Chapter 4. Tests performed for a total of 13 different batches of members from three different manufacturers were summarized and the issue of strain rate effects was discussed in light of the test results obtained.

In Chapters 5 through 8, the activities undertaken to establish each of the respective test sites are described along with a summary of observations from field performance monitoring of the sites to date. Activities for two test sites located in southern Kansas City and referred to as the I435-Kansas City sites were presented in Chapter 5. Activities at the US36-Stewartsville site located in northwest Missouri were described in Chapter 6. In Chapter 7, activities undertaken to establish three different test areas on I-70, at what is referred to as the I70-Emma site, were described. Finally, activities for the US54-Fulton test site in central Missouri were described in Chapter 8.

Finally, a series of implications that can be derived from the project to date were discussed in Chapter 9. This chapter included discussions of the overall effectiveness of the stabilization scheme, the apparent load transfer mechanisms determined from field performance monitoring, the overall suitability of the basic design methodology, the constructability of the stabilization measures, and the actual and expected costs of the stabilization measures. Additionally, several lessons learned regarding some of the field instrumentation utilized for the project are presented followed by a discussion of future implementation of the technique.

10.2. Conclusions

In any large project such as the one described in this report, numerous notable observations and conclusions can be made regarding many of the activities performed. While all of these observations and conclusions are important, some have limited significance in terms of the overall objectives of the project while others specifically relate to the overall objectives and have more far reaching implications. Observations and conclusions drawn from the project that have limited implications have been made within each of the respective chapters and will not be repeated here. However, several broad overall conclusions drawn from collective review of all project efforts warrant additional attention and are therefore summarized here. These broad conclusions include:

(1) Performance data collected from test sections established at the I70-Emma site during Phase I and the I435-Wornall Road site during Phase II over a period of more than three years indicates that surficial slope failures can be effectively stabilized using recycled plastic reinforcing members placed on a 3-ft by 3-ft (0.9-m by 0.9-m) staggered arrangement across the entire slide area.

(2) Performance data collected from additional tests sites established during Phase II indicate that surficial slope failures may possibly be stabilized using more widely spaced reinforcing members, with significant economic benefits. However, this conclusion must be tempered due to the fact that the remaining sites have yet to be subjected to conditions which are believed to have produced the original failures. Additional monitoring at these sites is critical to further evaluation of this possibility.

(3) Performance data collected from the field test sites indicate that each of the sites is following a consistent three-stage behavioral pattern. In the first stage, the stabilized slopes are observed to experience little movement and little resistance is provided by the reinforcing members. In Stage 2, slope movements are observed to increase substantially in response to increased pore water pressures within the slope at the same time as loads in the reinforcing members are observed to increase. These movements are believed to simply be movement required to mobilize resistance in the reinforcing members. Finally, Stage 3 is characterized by diminishing movement that is simultaneously accompanied by stabilization of the loads in the reinforcing members. This stage is believed to be a result of the slope and reinforcement coming to equilibrium. To date, test sections at the I70-Emma and I435-Wornall Road sites have exhibited all three stages of this behavioral pattern while the remaining test sections appear to be within the first or second stages of behavior.

(4) Some differences have been observed in the load transfer mechanisms at sites where reinforcing members were installed vertically and sites where members were installed perpendicular to the face of the slope. In general, members installed with a vertical orientation have been observed to mobilize lower axial loads and bending moments than members installed perpendicular to the slope face. However, additional monitoring and analysis is needed to confirm whether

these observations are consistent among all sites and stabilization schemes being evaluated. Furthermore, additional analysis is needed to establish how these different load transfer mechanism affect the overall stability of the slopes.

(5) Installation activities during Phase II have demonstrated that recycled plastic reinforcing members can be efficiently and reliably installed using either a percussion hammer found on many drilling rigs or a simple drop-weight type of hammer. Experience acquired to date has shown that the critical feature of installation equipment is having a mast to maintain the alignment between the hammer and the reinforcing member. As more experience has been acquired, and as better equipment has been utilized, the efficiency of installation has increased by a factor of almost two.

(6) Costs for stabilization of slopes using recycled plastic reinforcing members have been relatively consistent throughout the project. Nominal costs for materials and installation are approximately \$40/member with the costs being approximately equally split between material costs and installation costs. Unit costs per unit area of the slope face vary significantly with the spacing of the reinforcing members. Costs for stabilization using reinforcing members spaced at 3-ft (0.9-m) are nominally \$4.50/ft^2 (\$50/m^2). In contrast, costs for stabilization using reinforcing members spaced at 6-ft are nominally \$1.00/ft^2 (\$11/m^2). These observations place great importance on additional monitoring to establish whether surficial slope failures can be stabilized using more economical reinforcing schemes.

(7) The general design approach developed during Phase I and modified during Phase II continues to appear suitable for design of stabilization schemes using recycled plastic reinforcement. However, additional modifications are needed to evaluate the importance of axial forces in the reinforcing members and the model must be calibrated based on the performance observed at the field test sites.

(8) Evaluation and calibration of the design method would be greatly facilitated by the occurrence of a failure in one or more of the test sections established during the project. No such failures have occurred to date. However, several test sections with very low factors of safety are in place so it is hoped that one of these sections will fail once precipitation increases to normal or above normal levels.

10.3. Recommendations for Future Work

At present, the project has demonstrated the effectiveness of using recycled plastic reinforcement for stabilization of surficial slope failures. The method has also been shown to be constructible using equipment that is relatively commonplace. Extensive laboratory testing of recycled plastic members has enabled a good understanding of issues associated with the properties of the members to be developed and a draft specification for recycled plastic members for use in the slope stabilization application has been proposed. Based on these observations, it seems prudent to initiate implementation of the stabilization technique at additional sites on a trial basis. Such trials are expected to bring to light implementation issues that can otherwise not be foreseen, thereby facilitating development of an effective and timely plan for widespread implementation.

In addition, however, three general tasks remain to be addressed before widespread implementation can be effectively accomplished. These issues include:

(1) Establishing a correlation between field performance and stability (i.e. factor of safety or perhaps reliability) so that effective decisions can be made regarding whether to stabilize a particular site using recycled plastic reinforcement and, if so, what reinforcement pattern to utilize;

(2) Final modification and calibration of the design method and development of approximate design tools (e.g. charts, rules of thumb, etc); and

(3) Developing appropriate technology transfer materials so that appropriate personnel can be educated on appropriate procedures for design and construction of slope stabilization measures.

There are many details to each of these general tasks, which will require that a number of more specific tasks be performed. Several of the more detailed tasks are presented below to provide general direction for accomplishing the remaining general tasks:

10.3.1. Tasks to Establish Correlation between Field Performance and Stability

(1) Monitoring of existing field test sites should be continued so that future events leading to changes in the overall stability of the slopes can be accurately tracked and interpreted. Additional analysis and interpretation of such data are also necessary to update the results presented in this report.

(2) Additional techniques for estimating the mobilized lateral soil pressures from the reinforcing members should be evaluated to establish the actual stabilizing forces being provided by the reinforcing members. Such evaluations should make use of existing data from the instrumented reinforcing members as well as consideration of using deflections from the slope inclinometers in conjunction with software for analysis of laterally loaded piles (e.g. L-Pile®) for estimating these loads.

(3) Additional measures should be taken to facilitate accurate determination of the pore water pressures conditions within each of the stabilized slopes. These measures could include installation of sensors for directly measuring positive pore pressures on a continual basis at the I435-Wornall Road site, additional laboratory testing to establish soil-water characteristic curves so that moisture content data can be used to establish actual pore pressures, as well as additional evaluation of data obtained from the Profile Probe® to see if the scatter in the data can be reduced or appropriately considered.

(4) The general analysis methodology used for the project should be modified to incorporate consideration of axial forces in the reinforcing members. These capabilities will enable the importance and effects of axial forces measured in the instrumented reinforcing members to be evaluated.

(5) Additional analyses based on "as-built" conditions using measured pore water pressure conditions at specific points in time should be performed to relate theoretical stability with field performance. This task will require that as-built sections be established for each test section based on field installation data and that appropriate measures of performance be identified (e.g. deformations).

Analyses should then be performed using the as-built sections for selected field conditions determined from field instrumentation. The resulting factors of safety can then be plotted versus performance (e.g. deformation) to establish relations for each section, which can in turn be compared and contrasted to establish an overall relation for the technique.

10.3.2. Calibration and Final Modification of the Design Method

(1) Calibration will preferably be performed by matching theoretical factors of safety computed using the current design method with known factors of safety, preferably for several different slope stabilization schemes. Since factors of safety are only truly known at failure, this task will be greatly facilitated by the occurrence of one or more failures in the test sections established during Phase II. However, even if no failures occur, the method can still be calibrated by extrapolating the relation(s) between theoretical stability and performance described above to some failure criterion (e.g. limiting deformation) that is assumed to correspond to a factor of safety of unity.

(2) Based on results obtained from the above task, the design procedure should be modified as needed and finalized. Modifications are likely to include changes to the method used to predict the limiting soil pressures as well as possible incorporation of axial forces depending on the outcome of tasks discussed above. Modifications are not expected to involve changes to the general approach or procedure.

(3) Once the design method is finalized, a series of practical design tools should be developed for use by field personnel making decisions regarding slope repairs. While the exact form of these tools cannot be established at this time, consideration should be given to development of simple charts and/or tables, or even "rules-of-thumb" that provide measures of stability (i.e. factor of safety or reliability) for different possible stabilization schemes depending on the particular slope conditions present.

10.3.3. Development of Technology Transfer Materials

(1) A "Design and Construction Manual" should be developed to educate designers and field personnel about the technique. The document should include appropriate guidance for evaluation of the suitability of a site for stabilization with recycled plastic reinforcement, for selection of an appropriate stabilization scheme (e.g. design method or charts), for acquisition and evaluation of appropriate recycled plastic products (e.g. specification), and for efficient and reliable installation of the reinforcing members using either agency personnel or independent contractors.

(2) A short course should also be developed for the purpose of educating appropriate agency personnel and possible independent contractors about proper use of the technique. The short course should be initially offered at several locations around the state to facilitate rapid implementation of the technique. Additional offerings could then be made as the need arises.

(3) An "Applications Brief", possibly in electronic form (e.g. compact disc), should be developed for distribution to appropriate manufacturers of recycled plastic products. This document should include general information on the application as well as detailed information regarding the required properties of the recycled plastic members. Providing this information is critical to developing the manufacturer's understanding of what member characteristics are important (e.g. strength, stiffness) and what characteristics are not (e.g. aesthetics), which will ensure that the most cost effective members possible can be made available and may possibly lead to development of improved and/or less costly products in the future.

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Appendix A. Boring Logs for I435-Kansas City Sites

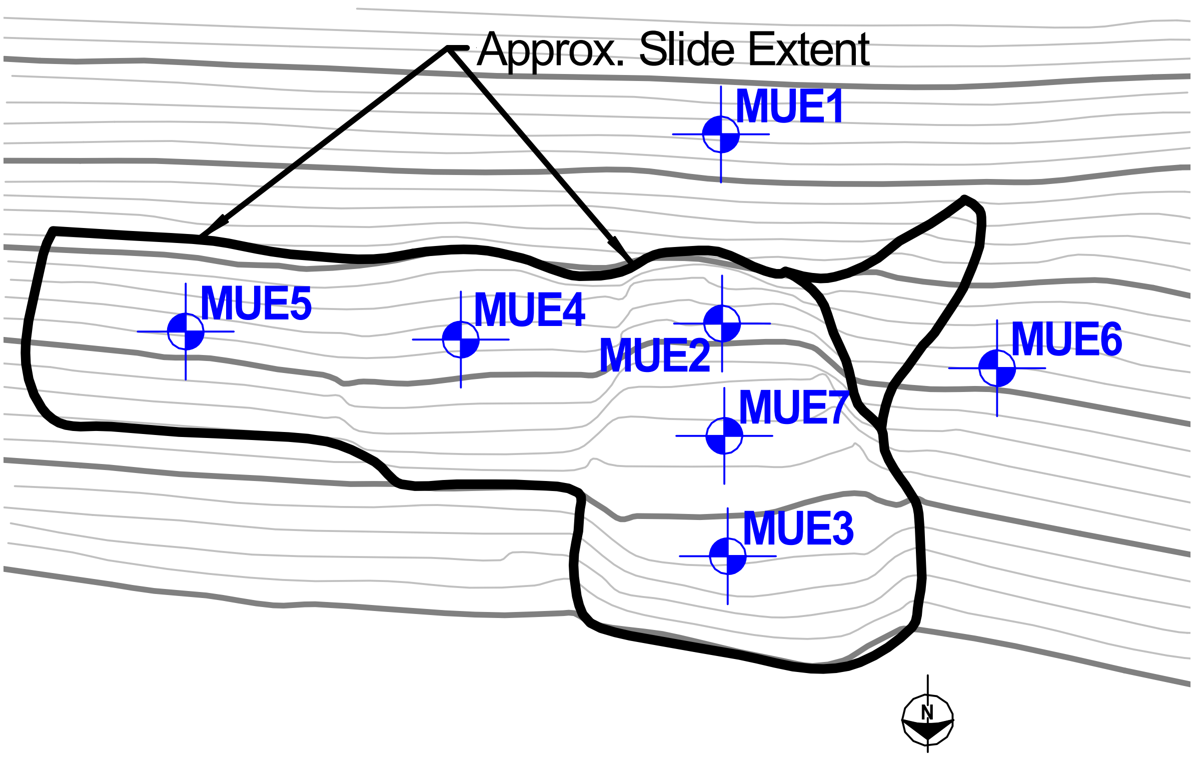

Figure A.1 Plan view of I435-Wornall Road site showing approximate boring locations.

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Project Name: I-435 - Kansas City Site | **Project NO: RI98-007B/SPRID55**

Location: I-435 & Wornall Road | **Boring Number: MUE1**

Ground Elevation: 846.0 | **Logged By: Fennessey**

Drilling Date: 6/26/2001 | **Driller: Murray**

Weather: Sunny, Hot | Auger Method: 4" Hollow Stem | Driller's Hole NO: Y-01-71

Conditions: | SPT Hammer Efficiency: | Drilling Equip: Versa-Drill G8641

Water Level: GWT not encountered

Soil Description	Sample NO	SPT N60	Moisture Content (%) (Including LL__PL)	Sample Type	Strength Test	Recovery (ft)
0.0-0.25 Mulch 0.25-2.9 Brown to red - brown lean clay, stiff, moist to wet, trace shale fragments	75			AL		0.65
	76			AL		0.45
2.9-18.0 Olive brown to yellow brown (shaley) fat clay, stiff to very stiff, moist, w/ shale fragments	77			AL		0.55
	78			3T		0.65
	79-80			3T-AL		0.75
	81-82			3T-AL		0.75
	83			AL		0.60
	84			3T		0.65
	85-86			3T-AL		0.80
	87-88			3T-AL		1.00
	89-90		29__19	3T-AL		0.80
18.0-19.0 Olive brown & tan mottled fat clay, very stiff, moist	91-92		76__25	3T-AL		0.75
19.0-20.0 Gravel layer (drilled)						
20.0-21.9 Olive brown to yellow brown (shaley) fat clay	93			3T		0.80
21.9-22.0 Brown clayey silt, stiff, moist 22.0-24.5 Olive brown fat clay, very stiff, moist	94-96		55__20	3T-AL-AL		1.00
24.5-31.5 Yellow brown fat to lean clay, stiff to very stiff, moist	97-98			3T-AL		1.00
	99-100			3T-AL		1.05
	101			AL		0.75
	102-103			3T-AL		0.90
31.5-32.3 Weathered limestone Boring completed at depth of 32.3						

AL--Sample taken for Atterberg Limits

CU--Consolidated Undrained Compression Test

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Project Name: I-435 - Kansas City Site	**Project NO: RI98-007B/SPRID55**
Location: I-435 & Wornall Road	**Boring Number: MUE2**
Ground Elevation: 838.2	**Logged By: Fennessey**
Drilling Date: 6/26/2001	**Driller: Murray**

Weather: Partly Cloudy, Hot	Auger Method: 4" Hollow Stem	Driller's Hole NO: Y-01-72
Conditions:	SPT Hammer Efficiency:	Drilling Equip: Versa-drill G8641

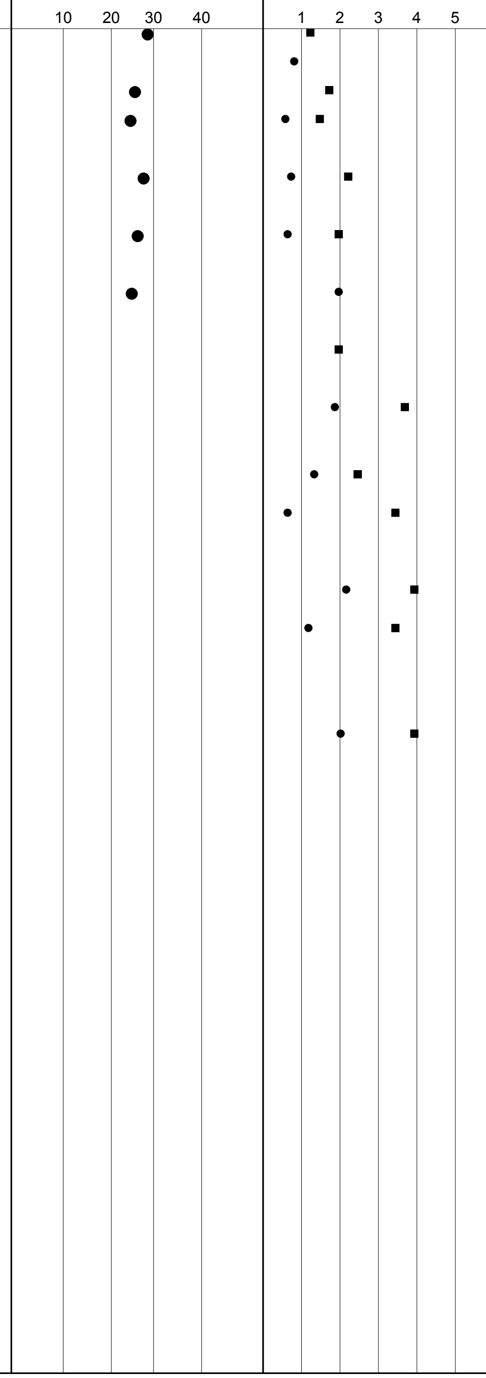
Water Level: GWT not encountered

Depth	Soil Description
0.0-0.4	Brown lean clay, stiff, moist
0.4-12.5	Yellow brown to olive brown (shaley) fat clay, stiff to very stiff, moist --trace shale fragments & concrete fragements --brown lean clay seam @2.1' --leaner (3.0-4.5) --with shale fragements & fat below 4.5'
12.5-13.0	Gray brown clayey silt to lean clay
13.0-19.0	Gray brown to red brown lean clay, very stiff to hard, moist --yellowish brown to olive brown below 15.3'
19.0-19.5	Limestone
	Boring completed at depth of 19.5

Sample NO	SPT N60	Sample Type	Strength Test	Recovery (ft)
104-105		AL-AL		0.80
106-107		3T-AL	CU	1.10
108-109		3T-AL	CU	1.15
110		AL		0.55
111-112		3T-AL	CU	1.05
113-114		3T-AL		0.80
115-116		3T-PL		0.95
117		PL		1.10
118-120		PL-3T-PL		1.35
121-123		3T-PL-PL		1.95
124-125		3T-PL		0.80
126-127		3T-PL		0.95

Moisture Content (%) (Including LL__PL)

Undrained Strength (TSF) TV ● PP ■



AL--Sample taken for Atterberg Limits

PL--Sample placed in plastic bag to preserve moisture

CU--Consolidated Undrained Compression Test

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Project Name: I-435 - Kansas City Site | **Project NO: RI98-007B/SPRID55**
Location: I-435 & Wornall Road | **Boring Number: MUE3**
Ground Elevation: 828.7 | **Logged By: Fennessey**
Drilling Date: 6/27/2001 | **Driller: Murray**

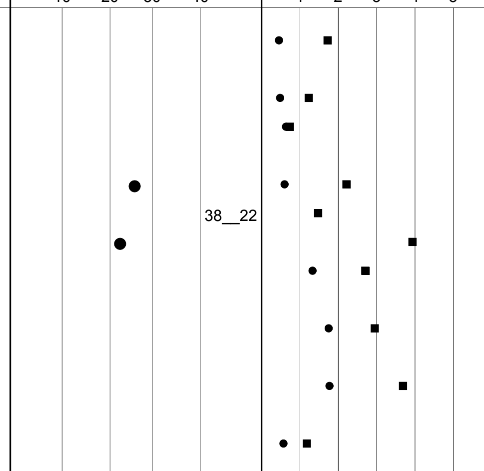
Weather: Sunny, Hot | Auger Method: 4" Hollow Stem | Driller's Hole NO: Y-01-74
Conditions: | SPT Hammer Efficiency: | Drilling Equip: Versa-drill G8641

Water Level: GWT not encountered

Depth (ft)	Soil Description
0.0-5.8	Brown to Olive brown, stiff to very stiff, lean clay, moist --mulch (0.0-0.2) --with shale & rock fragments below 1.5' --mulch @ 3.5' --mulch @ 4.9'
5.8-6.2	Brown clayey silt, hard, moist
6.2-12.0	Olive brown & red brown mottled lean clay --perched water @ 6.2' --becomes yellow brown to olive brown below 9.0'
12.0-12.4	Limestone
	Boring completed at depth of 12.4

Sample NO	SPT N60	Moisture Content (%) (Including LL__PL)	Sample Type	Strength Test	Recovery (ft)
139			PL		0.65
140			PL		0.35
141			PL		0.65
142-144		38__22	3T-PL-PL	CU(2)	1.20
145-146			3T-PL		1.00
147-148			3T-PL		0.85
149			PL		0.70
150-151			3T-PL		0.90

Moisture Content (%) (Including LL__PL): 10 20 30 40
Undrained Strength (TSF): TV ● PP ■ — 1 2 3 4 5



Elevation: 0, -5, -10, -15, -20, -25, -30, -35
Depth: 0, 5, 10, 15, 20, 25, 30, 35

PL--Sample placed in plastic bag to preserve moisture
CU--Consolidated Undrained Compression Test

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Project Name: I-435 - Kansas City Site	**Project NO: RI98-007B/SPRID55**
Location: I-435 & Wornall Road	**Boring Number: MUE4**
Ground Elevation: 837.7	**Logged By: Fennessey**
Drilling Date: 6/26/2001	**Driller: Murray**

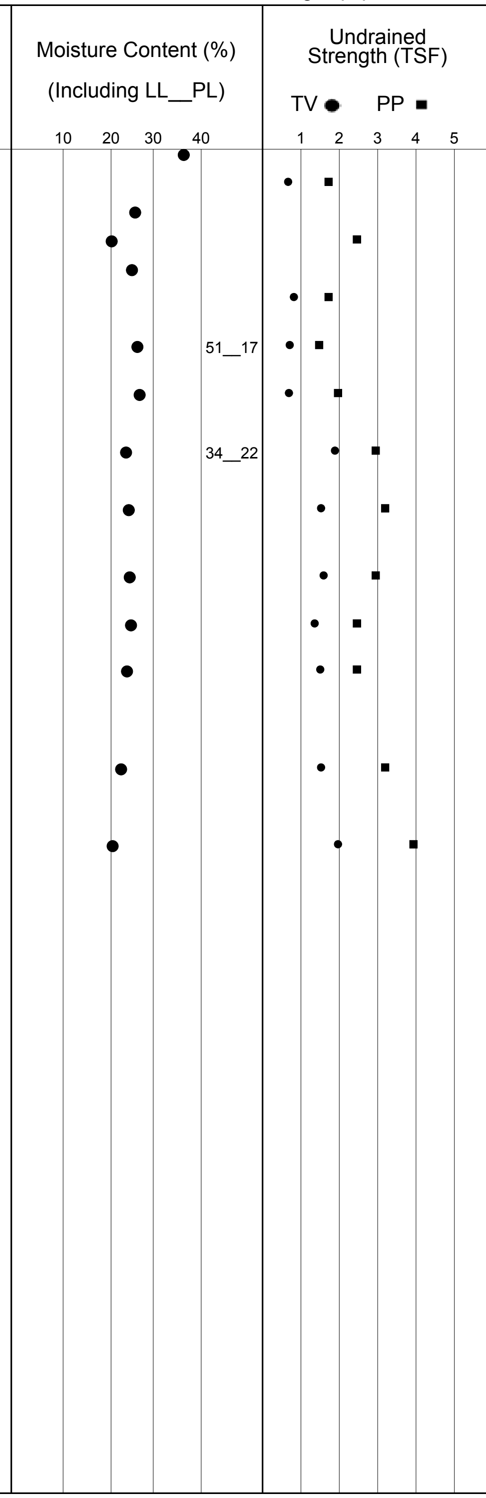
Weather: Sunny, Warm	Auger Method: 4" Hollow Stem	Driller's Hole NO: Y-01-70
Conditions:	SPT Hammer Efficiency:	Drilling Equip: Versa-drill G8641

Water Level: GWT not encountered

Elevation: 0, -5, -10, -15, -20, -25, -30, -35

Depth: 0, 5, 10, 15, 20, 25, 30, 35

Soil Description	Sample NO	SPT N60	Moisture Content (%) (Including LL__PL)	Sample Type	Strength Test	Recovery (ft)
0.0-0.4 Brown lean clay, soft, wet 0.4-3.0 Yellow brown to olive brown (shaley) fat clay, stiff to very stiff, moist	54-55			AL-AL		0.7
	56			3T		0.8
3.0-3.2 Brown lean clay, stiff, moist 3.2-13.0 Yellow brown to tan (shaley) fat clay, stiff to very stiff, moist, with shale fragments and nodules --olive brown to yellow brown below 7.0'	57-59			AL-3T-AL		0.8
	60-61		51__17	3T-AL	CU	1.05
	62-63			3T-AL		0.9
	64-65		34__22	3T-AL	CU	1.15
	66-67			3T-AL		1.05
	68			AL		0.75
	69			AL		0.50
13.0-14.2 Brown to yellow brown fat clay, very stiff, moist --concrete fragments @ 13.9	70			AL		0.45
14.2-19.0 Olive brown to yellow brown fat to lean clay, very stiff to hard, moist --natural ground @ 14.2 +/- ft.	71-72			3T-AL		0.80
	73-74			3T-AL		1.2
19.0-19.6 Limestone --weathered (19.0-19.1) Boring completed at depth of 19.6						



3T refusal @ 12.5' --cleaned out to 13.0'

3T refusal @ 13.9' --cleaned out to 15.0'

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Project Name: I-435 - Kansas City Site | **Project NO: RI98-007B/SPRID55**

Location: I-435 & Wornall Road | **Boring Number: MUE5**

Ground Elevation: 837.5 | **Logged By: Fennessey**

Drilling Date: 6/25/2001 | **Driller: Murray**

Weather: Sunny & Hot | Auger Method: 4" Hollow Stem | Driller's Hole NO: Y-01-69

Conditions: | SPT Hammer Efficiency: | Drilling Equip: Versa-drill G8641

Water Level: GWT not encountered

Depth	Soil Description	Sample NO	SPT N60	Moisture Content (%) (Including LL__PL)	Undrained Strength (TSF) TV ● PP ■	Sample Type	Strength Test	Recovery (ft)
0.0-0.2	Mulch							
0.2-2.4	Brown lean clay, medium stiff, moist	38-39				3T-AL	CU(2)	1.05
2.4-6.5	Yellow brown (shaley) fat clay, medium stiff to very stiff, moist --obstruction (cobble?) @ 5.8'	40-41				3T-AL		1.10
		42-43				3T-AL		1.25
6.5-13.7	Gray brown (shaley) fat clay, stiff to very stiff, moist, with shale fragments --slickensided shale fragment @ 8.0'	44				AL		0.85
		45-46				3T-AL		1.50
		47				AL		0.65
13.7-17.6	Brown fat to lean clay, very stiff, moist, possible original ground, trace rock fibers @ 13.7'	48-49		37__19		3T-AL		1.20
		50-51		37__21		3T-AL		1.20
17.6-18.5	Yellow brown fat clay, stiff to very stiff, moist	52-53		56__18		3T-AL		0.90
18.5-19.3	Limestone							
	Boring completed at depth of 19.3							

damaged 3T (4.0-6.0) -- drilled out obstruction from 6.0-6.5

--drilled out obstruction from 8.5-9.5

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Project Name: I-435 - Kansas City Site	**Project NO: RI98-007B/SPRID55**
Location: I-435 & Wornall Road	**Boring Number: MUE6**
Ground Elevation: 832.7	**Logged By: Fennessey**
Drilling Date: 6/27/2001	**Driller: Murray**

Weather: Overcast, showers, and sunny	Auger Method: 4" Hollow Stem	Driller's Hole NO: Y-01-75
Conditions:	SPT Hammer Efficiency:	Drilling Equip: Versa-drill G8641

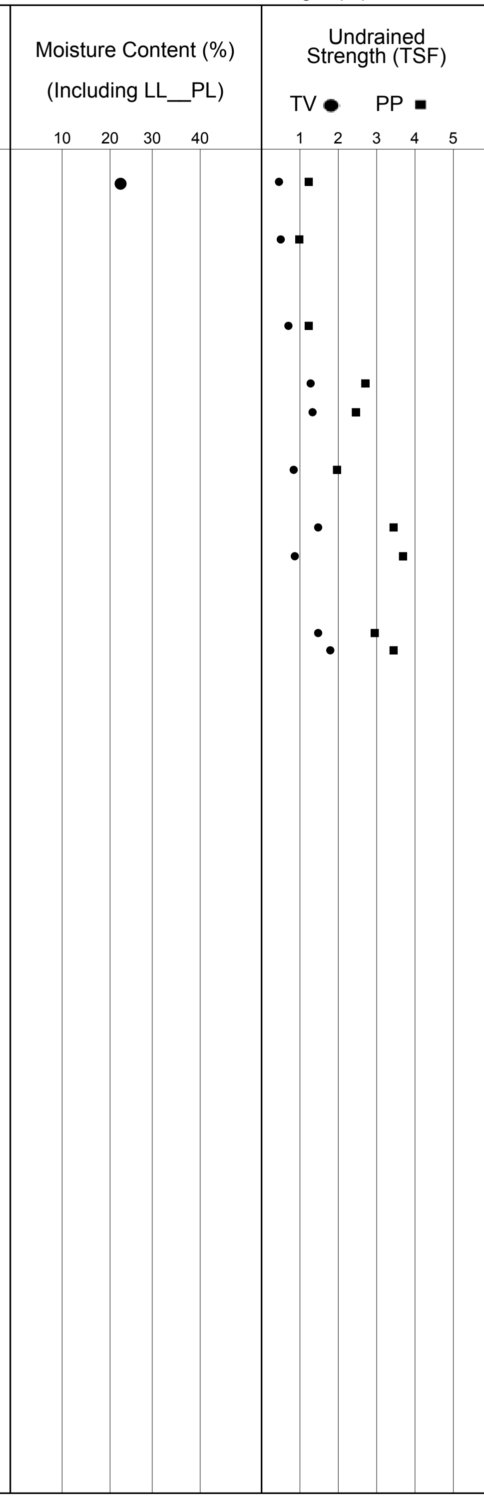
Water Level: GWT not encountered

Soil Description

- 0.0-4.5 Brown lean clay, stiff, moist --mulch (0.0-0.2)
- 4.5-6.0 Yellow brown lean clay, stiff to very stiff, moist --becoming more plastic & stiffer w/ depth
- 6.0-7.7 Yellow brown to olive brown (shaley) fat clay, very stiff, moist --with shale fragments
- 7.7-9.0 Brown to gray brown lean clay, very stiff, moist --clayey silt (7.7-8.0) --trace organics @ 7.7'
- 9.0-11.7 Gray brown & red brown mottled fat to lean clay, very stiff, moist
- 11.7-13.4 Yellow brown lean clay, very stiff, moist --trace gravel (12.5-13.4)
- 13.4-13.9 Limestone

Boring completed at depth of 13.9

Sample NO	SPT N60	Sample Type	Strength Test	Recovery (ft)
152-153		3T-PL	CU	0.80
154		PL		0.25
				0
155-157		3T-PL-PL		1.05
158-159		3T-PL		1.05
160-161		3T-PL		0.9
162-163		3T-PL		1.15
164-167		3T-PL-3T-PL		1.80
168		3T		0.85



PL--Sample placed in plastic bag to preserve moisture

CU--Consolidated Undrained Compression Test

No sample recovered (3.0-4.5)

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Project Name: I-435 - Kansas City Site	**Project NO: RI98-007B/SPRID55**
Location: I-435 & Wornall Road	**Boring Number: MUE7**
Ground Elevation: 831.3	**Logged By: Fennessey**
Drilling Date: 6/27/2001	**Driller: Murray**

Weather: Sunny, Warm	Auger Method: 4" Hollow Stem	Driller's Hole NO: Y-01-73
Conditions:	SPT Hammer Efficiency:	Drilling Equip: Versa-drill G8641

Water Level: GWT not encountered

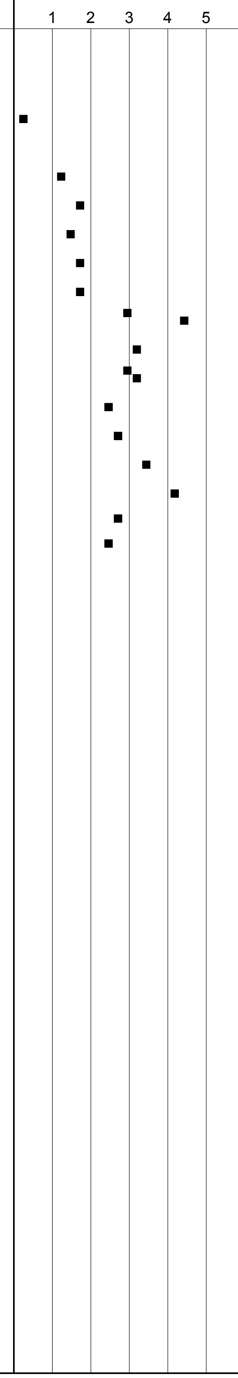
Depth	Soil Description
0.0-3.2	Brown lean clay, very soft to soft, moist
3.2-7.3	Yellow brown to olive brown (shaley) fat clay, stiff to very stiff, moist (with shale fragments)
7.3-8.8	Brown lean clay, very stiff to hard, moist -- (becoming fatter & more moist w/ depth)
8.8-10.6	Gray brown & red brown mottled lean to fat clay -- (olive gray fat clay seam 9.0-9.1)
10.6-13.1	Yellow brown, very stiff to hard lean clay, moist -- (becoming fatter & more moist w/ depth) -- (fat clay 12.7-13.1)
13.1-13.8	Limestone
	Boring completed at depth of 13.8

Sample NO	SPT N60	Sample Type	Recovery (ft)
	WH	SS	0.0
128	WH	SS	0.3
129-130	*/*/2	SS	0.85
131	*/2/4	SS	1.00
132	1/3/5	SS	1.35
133-134	8/7/9	SS	1.50
135	3/4/7	SS	1.00
136	3/5/7	SS	1.50
137-138	2/7/40**	SS	1.00

Moisture Content (%) (Including LL__PL): 10 20 30 40

Strength Test

Undrained Strength (TSF) TV ● PP ■



WH--Sampler pushed 18" by the weight of the hammer

* Sampler pushed 6" by the weight of the hammer

** The 40 blows only moved the sampler 3.5"

Plastic bag samples were also taken from each 1.5' increment

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Project Name: I-435 - Kansas City Site | **Project NO: RI98-007B/SPRID55**

Location: I-435 & Wornall Road | **Boring Number: Control Slope SW Quadrant**

Ground Elevation: 8' above toe of slope | **Logged By: B. Temme**

Drilling Date: 7/10/2002 | **Driller: K. Barnett**

Weather: Sunny 85 deg | Auger Method: 4" Hollow Stem | Driller's Hole NO: B-02-56

Conditions: | SPT Hammer Efficiency: | Drilling Equip: Versa drill G8690

Water Level: GWT not encountered

Depth (ft)	Soil Description	Sample NO	SPT N60	Moisture Content (%) (Including LL__PL)	Sample Type	Strength Test	Recovery (ft)
0.0-7.1	Light brown lean clay, trace gravel, moist, stiff to hard	24-25		42__21	3T-AL		1.4
		26-27		55__26	3T-AL		1.2
		28-29			3T-AL		0.9
7.1-12.9	Brown and gray shaley clay, moist, hard to very stiff	30-31		40__21	3T-AL		1.3
		32-33			3T-AL		2.1
12.9-18.3	Dark gray glacial till, very stiff, moist -- (hard shale @ 18.3)	34-35			3T-AL		1.5
		36-37			3T-AL		1.9
		38-39			3T-AL		0.8
	Boring completed at depth of 18.3						

Undrained Strength (TSF): TV ● PP ■

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Project Name: I-435 Kansas City Site | **Project NO: RI98-007B**

Location: I-435 & Holmes Road | **Boring Number: Steel Pin Slope -- SE Quadrant**

Ground Elevation: 10' above toe, offset 6' downslope | **Logged By: R. Temme**

Drilling Date: 7/11/2002 | **Driller: K. Barnett**

Weather: Overcast, 70 deg | Auger Method: 4" Hollow Stem | Driller's Hole NO: B-02-57

Conditions: | SPT Hammer Efficiency: | Drilling Equip: Versa drill G8690

Elevation	Profile	Water Level	Depth	Soil Description	Sample NO	SPT N60	Moisture Content (%) (Including LL__PL) 10 20 30 40	Undrained Strength (TSF) TV ● PP ■ 1 2 3 4 5	Sample Type	Strength Test	Recovery (ft)
0		GWT not encountered	0	0.0-4.3 Brown lean clay, medium stiff, moist	40-41			■ ●	3T-AL		1.1
					42-43			■ ●	3T-AL		2.1
-5			5	4.3-19.0 Gray shale, very soft, moist -- (refusal @ 5.3')	44				AL		0.3
						14/27/40* 100+			SS		1.0
						27/40** 100+			SS		0.3
-10			10								
-15			15								
-20			20	Boring completed at depth of 19.0							
-25			25								
-30			30								
-35			35								

SPT corrected N60 values given below blow sequence

* No advance

** Advanced only 2"

Appendix B. Boring Logs for US36-Stewartsville Site

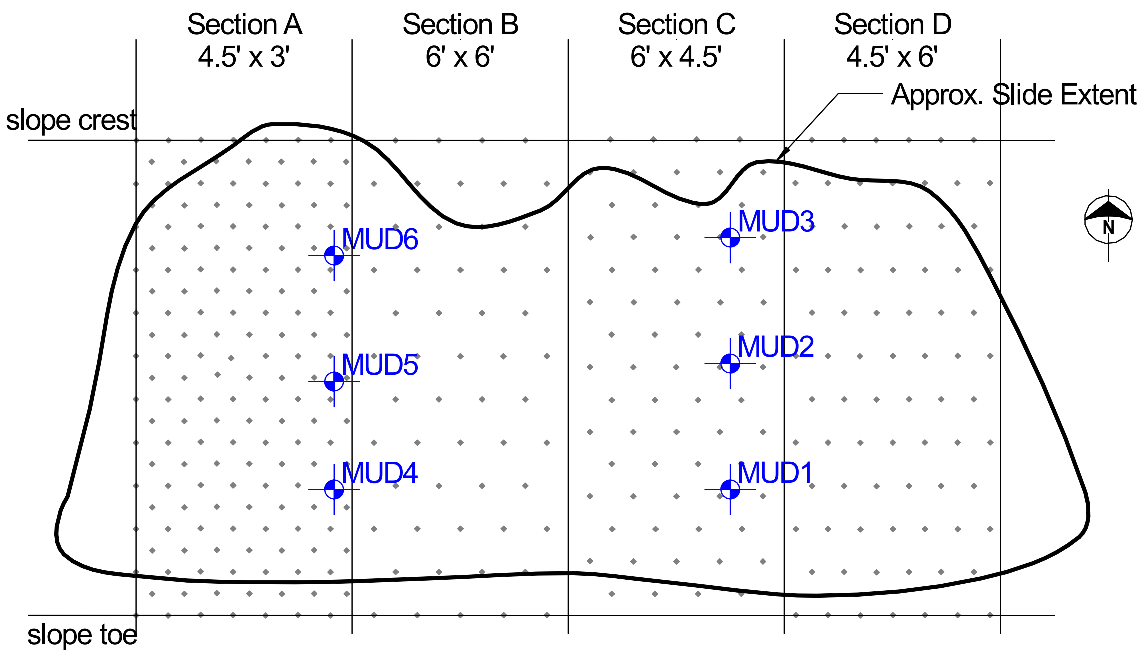

Figure B.1 Plan view of US36-Stewartsville site showing approximate boring locations.

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Project Name: US-36 Stewartsville Site	**Project NO: SPROID5S**
Location: Between eastbound & westbound US-36	**Boring Number: MUD1**
Ground Elevation:	**Logged By: Less**
Drilling Date: 5-30-2001, 6/4/2001	**Driller: Murray/Hees**

Weather: Cloudy, Raining	Auger Method: 4" Hollow Stem	Driller's Hole NO: Y-01-55
Conditions:	SPT Hammer Efficiency:	Drilling Equip: Versa Drill 4000TR-2 G8641

Water Level: GWT not encountered

Depth (ft)	Soil Description	Sample NO	SPT N60	Moisture Content (%) (Including LL__PL)	Sample Type	Strength Test	Recovery (ft)
0.0-2.5	Brown to gray fat mottled clay, medium stiff, moist	275-276		45__16	AL-3T	CU	1.1
2.5-4.5	Gray fat clay, trace gravel, medium stiff, moist	277-278		46__21	AL-3T	CU	1.7
4.5-10.5	Gray to brown mottled lean clay, brittle, medium stiff to hard	279-280		40__19	AL-3T	CU	1.9
		281-282			AL-3T	CU	1.7
		283-284			AL-3T		1.7
10.5-15.1	Tan lean clay with gray mottling, trace gravel, stiff to very stiff, moist --Refusal 12.4-12.9, cleaned with auger	285-286			AL-3T		1.9
		287-288			AL-3T*		0.5
		289-290			AL-3T		1.2
		291-292			AL-3T		1.0
	Boring completed at depth of 15.1						

Note: 5/30/2001 lost tube in hole @ 11.6' -- moved over to the east 2.0' on 6/4/2001 & resumed drilling back at 10.5'

* No sample recovered

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Project Name: US-36 Stewartsville Site | **Project NO: SPROID5S**

Location: Between eastbound & westbound US-36 | **Boring Number: MUD2**

Ground Elevation: | **Logged By: Less**

Drilling Date: 6/5/2001 | **Driller: Murray**

Weather: Cloudy 70-75 deg | Auger Method: 4" Hollow Stem | Driller's Hole NO: Y-01-57

Conditions: | SPT Hammer Efficiency: | Drilling Equip: Versa Drill 4000TR-2 G8641

Water Level: GWT not encountered

Depth	Soil Description	Sample NO	SPT N60	Moisture Content (%) (Including LL__PL)	Undrained Strength (TSF) TV ● PP ■	Sample Type	Strength Test	Recovery (ft)
0.0-2.5	Tan to gray fat clay, soft to medium stiff, moist	315-316		69__24		AL-3T	CU	1.1
2.5-7.5	Gray fat clay, medium stiff, moist	317-318		52__21		AL-3T	CU	1.7
		319-320		65__24		AL-3T		2.0
7.5-9.0	Gray lean clay, medium stiff to stiff, moist	321-322		51__20		AL-3T	CU	2.0
9.0-11.7	Tan to gray lean clay with gravel, stiff to very stiff, moist	323-324		41__20		AL-3T	CU	1.8
11.7-14.0	Gray to tan lean clay, mottled with gravel, moist	325-326				AL-3T		1.9
		327-328				AL-3T		1.3
14.0-20.5	Tan to brown lean clay, trace gravel, moist, very stiff	329-330				AL-3T		1.5
		331-332				AL-3T		2.0
		333-334				AL-3T		1.5
		335-336				AL-3T		1.5

Boring completed at depth of 20.5

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Project Name: US-36 Stewartsville Site | **Project NO: SPROID5S**

Location: Between eastbound & westbound US-36 | **Boring Number: MUD3**

Ground Elevation: | **Logged By: Less**

Drilling Date: 6-4-2001-6/5/2001 | **Driller: Hees/Murray**

Weather: Cloudy, Windy, Cool, 68 deg | Auger Method: 4" Hollow Stem | Driller's Hole NO: Y-01-56

Conditions: | SPT Hammer Efficiency: | Drilling Equip: Versa Drill 4000TR-2 G8641

Water Level: GWT not encountered

Depth	Soil Description	Sample NO	SPT N60	Moisture Content (%) (Including LL__PL)	Sample Type	Strength Test	Recovery (ft)
0	0.0-2.5 Gray to tan mottled fat clay, medium stiff, moist -- sample compacted	295-296			AL-3T*		1.3
	2.5-12.5 Gray to tan mottled lean clay, stiff to very stiff, moist with gravel	297-298			AL-3T		1.7
5		299-300		33__26 37__19	AL-3T	CU	2.0
		301-302		44__18	AL-3T	CU	2.5
10		303-304			AL-3T		2.2
	12.5-20.2 Brown to gray mottled lean clay, stiff to very stiff with gravel, moist		7/9/15		SS		1.5
15		305-306		44__16	AL-3T		1.3
			6/9/15		SS		1.5
		307-308			AL-3T		1.2
			7/8/17		SS		1.5
20	20.2-21.9 Pushed shelby tube, bent tube, no sample, hit cobble				3T*		
	21.9-25.0 Brown to gray lean clay, trace gravel, very stiff, moist -- dent in sample from 21.9-23.2	309-310			AL-3T**		1.3
		311-312			AL-3T		1.8
25	Boring completed at depth of 25.0						

* Shelby tube refusal -- no sample

** Damaged shelby tube sample

SuperLog V3.0A CivilTech Software, USA www.civiltech.com File: C:\superlog3\project\US-36_boring.log Date: 10/16/2003

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Project Name: US-36 Stewartsville Site	**Project NO: SPROID5S**
Location: Between eastbound & westbound US-36	**Boring Number: MUD4**
Ground Elevation:	**Logged By: Less**
Drilling Date: 6/6/2001	**Driller: Murray**

Weather: Cloudy, Rainy, 68 deg	Auger Method: 4" Hollow Stem	Driller's Hole NO: Y-01-60
Conditions:	SPT Hammer Efficiency:	Drilling Equip: Versa Drill 4000TR-2 G8641

Elevation	Profile	Water Level	Depth	Soil Description	Sample NO	SPT N60	Moisture Content (%) (Including LL__PL) 10 20 30 40	Undrained Strength (TSF) TV ● PP ■ 1 2 3 4 5	Sample Type	Strength Test	Recovery (ft)
0		GWT not encountered	0	0.0-2.5 Tan to gray to brown lean clay, soft, moist, (sample compacted)	370-371		48__25		AL-3T		1.2
				2.5-5.0 Tan to gray lean clay, soft, moist with gravel (sample compacted)	372-373				AL-3T		1.3
-5			5	5.0-15.5 Tan to gray lean clay, with gravel, very stiff, moist	374-375				AL-3T	CD	2.2
					376-377				AL-3T		1.5
-10			10		378-379				AL-3T		1.5
					380-381				AL-3T		1.9
					382-383				AL-3T		1.5
-15			15		384-385				AL-3T		1.4
				Boring completed at depth of 15.5							
-20			20								
-25			25								
-30			30								
-35			35								

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Project Name: US-36 Stewartsville Site	**Project NO: SPROID5S**
Location: Between eastbound & westbound US-36	**Boring Number: MUD5**
Ground Elevation:	**Logged By: Less**
Drilling Date: 6/6/2001	**Driller: Murray**

Weather:	Auger Method: 4" Hollow Stem	Driller's Hole NO: Y-01-59
Conditions:	SPT Hammer Efficiency: 75	Drilling Equip: Versa Drill 4000TR-2 G8641

Water Level: GWT not encountered

Soil Description
0.0-2.5 Brown to gray mottled fat clay, soft, moist
2.5-9.0 Brown to gray lean clay with gravel, stiff to very stiff, moist
9.0-10.5 Tan to gray lean clay with gravel, moist
10.5-13.5 Tan to gray lean clay, sandy, with gravel, moist
13.5-15.0 Tan to gray lean clay, with gravel, very stiff, moist
15.0-16.5 Brown lean clay, very stiff, moist
16.5-18.0 Brown to gray mottled lean clay, very stiff, moist
18.0-21.0 Brown lean clay, sandy, trace gravel, moist
Boring completed at depth of 21.0

Sample NO	SPT N60	Sample Type	Strength Test	Recovery (ft)
	WH	SS		
	WH/1	SS		
	2/3/4 9	SS		
	2/3/6 11	SS		
	3/7/9 20	SS		
	4/5/8 16	SS		
	3/3/7 13	SS		
	3/6/9 19	SS		
	3/6/8 18	SS		
	3/6/8 18	SS		
	4/6/8 18	SS		
	4/7/9 20	SS		
	3/6/8 18	SS		
	4/5/8 16	SS		

Elevation: 0, -5, -10, -15, -20, -25, -30, -35

Depth: 0, 5, 10, 15, 20, 25, 30, 35

Profile

Moisture Content (%) (Including LL__PL): 10, 20, 30, 40

Undrained Strength (TSF): TV ● PP ■ : 1, 2, 3, 4, 5

WH -- Weight of hammer penetrated soil

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Project Name: US-36 Stewartsville Site	**Project NO: SPROID5S**
Location: Between eastbound & westbound US-36	**Boring Number: MUD6**
Ground Elevation:	**Logged By: Less**
Drilling Date: 6/5/2001	**Driller: Murray**

Weather: Sunny, Warm 75-80 deg — Auger Method: 4" Hollow Stem — Driller's Hole NO: Y-01-58

Conditions: — SPT Hammer Efficiency: — Drilling Equip: Versa Drill 4000TR-2 G8641

Water Level: GWT not encountered

Depth	Soil Description	Sample NO	SPT N60	LL__PL	Sample Type	Strength Test	Recovery (ft)
0.0-2.5	Tan lean clay, soft to medium stiff, moist -- sample compacting	340-341		37__22	AL-3T*		1.3
2.5-5.0	Tan to gray fat clay, soft, moist	342-343		53__18 39__20	AL-3T	CD(2)	1.3
5.0-7.5	Tan lean clay, trace gravel, medium stiff, moist	344-345			AL-3T		2.0
7.5-9.0	Brown to gray sandy lean clay, very stiff, moist, trace gravel	346-347		44__20	AL-3T		1.5
9.0-12.5	Gray lean clay, trace gravel, stiff to very stiff, moist	348-349			AL-3T		1.5
		350-351		44__20	AL-3T		1.8
12.5-25.0	Brown to gray mottled lean clay, very stiff, moist, with gravel	352-353		55__21	AL-3T		1.5
		354-355			AL-3T*		1.5
		356-357		45__19	AL-3T		1.8
		358-359			AL-3T		2.0
		360-361			AL-3T		2.2
		362-363			AL-3T		1.5
		364-365			AL-3T		1.2
25.0	Boring completed at depth of 25.0						

* Shelby Tube damaged --- no sample

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Project Name: US-36 Stewartsville Site | **Project NO: SPROID5S**

Location: Between eastbound & westbound US-36 | **Boring Number: MUD7**

Ground Elevation: | **Logged By: Less**

Drilling Date: 6/6/2001 | **Driller: Murray**

Weather: Sunny 80-85 deg | Auger Method: 4" Hollow Stem | Driller's Hole NO: Y-01-61

Conditions: | SPT Hammer Efficiency: | Drilling Equip: Versa Drill G8641

Water Level: GWT not encountered

Depth	Soil Description	Sample NO	SPT N60	Moisture Content (%) (Including LL__PL)	Sample Type	Strength Test	Recovery (ft)
0	0.0-5.0 Gray to tan lean clay, soft, moist -- (sample compacted)	390-391			AL-3T*		0.9
		392-393		47__20	AL-3T		1.8
5	5.0-12.0 Gray to tan lean clay, medium stiff to stiff to very stiff, moist, with gravel	394-395			AL-3T		1.9
		396-697		46__20	AL-3T		1.4
		398-399		43__16	AL-3T		1.3
10		400-401			AL-3T		1.9
12	12.0-15.5 Tan lean clay, with gravel, very stiff, moist	402-403			AL-3T		1.7
15		404-405			AL-3T		1.5
15.5	15.5-20.0 Tan to gray mottled lean sandy clay, very stiff, moist	406-407			AL-3T		2.0
		408-409			AL-3T		1.3
20	Boring completed at depth of 20.0						

Moisture Content scale: 10 20 30 40

Undrained Strength (TSF): TV ● PP ■ — scale 1 2 3 4 5

Elevation / Depth scale: 0, -5, -10, -15, -20, -25, -30, -35 / 0, 5, 10, 15, 20, 25, 30, 35

* Tube damaged, no sample

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Project Name: US-36 Stewartsville Site
Project NO: SPROID5S
Location: Between eastbound & westbound US-36
Boring Number: MUD8
Ground Elevation:
Logged By: Less
Drilling Date: 6/7/2001
Driller: Murray

Weather: Cloudy, Humid, 70 deg
Auger Method: 4" Hollow Stem
Driller's Hole NO: Y-01-62
Conditions:
SPT Hammer Efficiency:
Drilling Equip: Versa Drill 4000TR-2 G8641

Water Level: GWT not encountered

Depth (ft)	Soil Description	Sample NO	SPT N60	Moisture Content (%) (Including LL__PL)	Undrained Strength (TSF) TV ● PP ■	Sample Type	Strength Test	Recovery (ft)
0.0-2.5	Brown to gray fat clay, soft, moist	415-416		58__23		AL-3T		1.2
2.5-7.0	Gray to tan mottled lean clay, with gravel, stiff to very stiff, moist	417-418				AL-3T		1.9
		419-420				AL-3T		1.5
7.0-10.0	Tan to gray mottled lean clay, trace gravel, very stiff, moist	421-422		47__21		AL-3T		1.3
		423-424				AL-3T		1.4
10.0	Boring completed at depth of 10.0							

Elevation scale: 0, -5, -10, -15, -20, -25, -30, -35
Depth scale: 0, 5, 10, 15, 20, 25, 30, 35
Moisture Content scale: 10, 20, 30, 40
Undrained Strength scale: 1, 2, 3, 4, 5

Appendix C. Boring Logs for I70-Emma Site

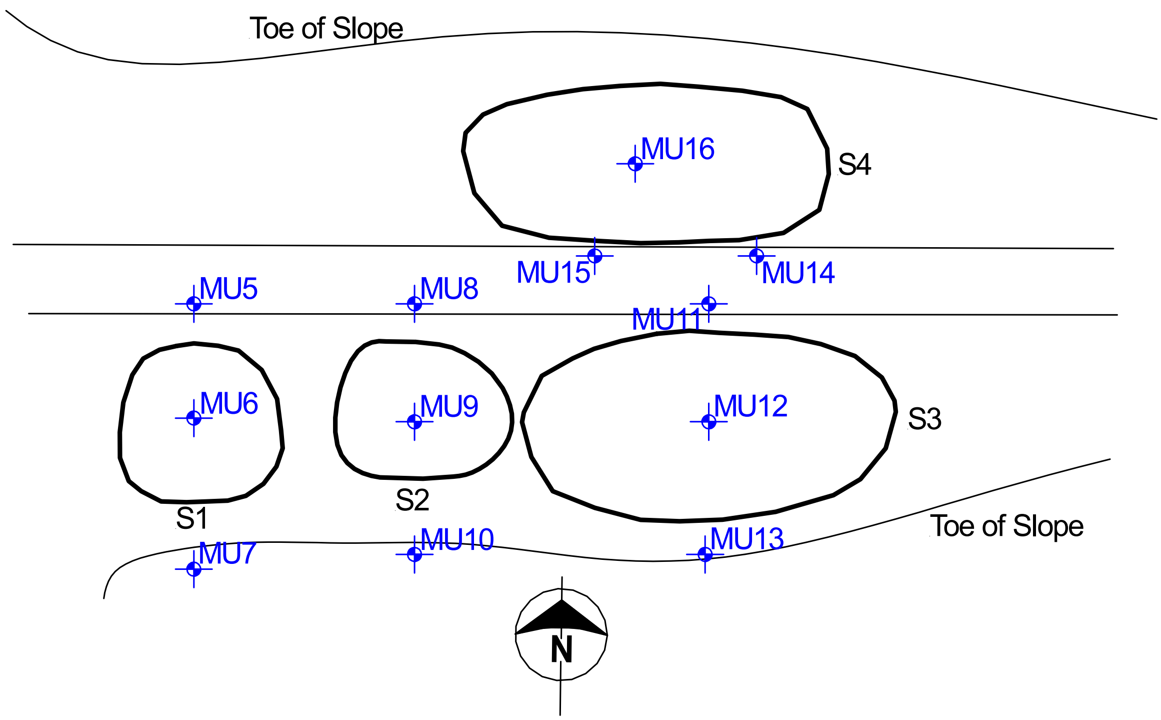

Figure C.1 Plan view of I70-Emma site showing approximate boring locations.

SuperLog V3.0A CivilTech Software, USA www.civiltech.com File: C:\superlog3\project\EMMA_boring.log Date: 10/16/2003

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Project Name: I-70 - Emma Site | **Project NO: Test 7**

Location: Eastbound on-ramp @ Jct. Rt. Y & I-70 | **Boring Number: MU5**

Ground Elevation: | **Logged By: A. Miller**

Drilling Date: 6/1/1999 | **Driller: Lamberson**

Weather: Sunny | Auger Method: | Driller's Hole NO: L-99-24

Conditions: | SPT Hammer Efficiency: | Drilling Equip: Failing 1500 G7889

Water Level: 6/2/99, 6/3/99

Soil Description
0.0-3.0 Asphalt & base rock
3.0-10.5 Brown & gray fat clay, stiff -- trace fine gravel beginning @ 10'
10.5-15.2 Brown lean clay, trace fine gravel, stiff to very stiff, some lignite
15.2-15.5 Dark brown lean clay, stiff
15.5-16.2 Gray & brown fat clay, stiff
16.2-16.5 Black tar paper, possible ditch liner
16.5-22.4 Gray & brown lean clay, scattered fine gravel, very stiff
22.4-23.0 Dark brown lean clay, very stiff, scattered gravel
23.0-28.0 Brown & gray fat clay, trace fine gravel, stiff
28.0-33.0 Reddish brown gray mottled lean to fat clay, very stiff, scattered gravel
Boring completed at depth of 33.0

Sample NO	SPT N60	Moisture Content (%) (Including LL__PL)	Sample Type	Strength Test	Recovery (ft)
			3T		0
100-102		56__21	3T		1.5
			3T		2.5
107-109			3T		2.5
110-112			3T		2.0
114			3T		1.0
113,115			3T		1.0
116-119			3T		2.5
120			3T		1.0
121			3T		0.6
122-124			3T		1.5
125-126			3T		1.2

Undrained Strength (TSF): TV ● PP ■ (scale 1 2 3 4 5)

Moisture Content scale: 10 20 30 40

Elevation / Depth: 0 to -35 / 0 to 35

--water table artificially high due to drilling method

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Project Name: I-70 - Emma Site | **Project NO: Test 7**
Location: Eastbound on-ramp @ Jct. Rt. Y & I-70 | **Boring Number: MU6**
Ground Elevation: | **Logged By: P. Hilchen**
Drilling Date: 6/1/1999 | **Driller: Varnes**

Weather: Overcast with occasional showers | Auger Method: | Driller's Hole NO: Y-99-38
Conditions: | SPT Hammer Efficiency: | Drilling Equip: Sonco 4000

Column headings: Elevation | Profile | Water Level | Depth | Soil Description | Sample NO | SPT N60 | Moisture Content (%) (Including LL__PL) 10 20 30 40 | Undrained Strength (TSF) TV ● PP ■ 1 2 3 4 5 | Sample Type | Strength Test | Recovery (ft)

Soil Description	Sample NO	SPT N60	LL__PL	Sample Type	Strength Test	Recovery (ft)
0.0-3.0 Brown lean clay with scattered gravel, hard	157-159			3T		2.1
3.0-4.5 Dark brown lean clay, stiff	160-162		39__26	3T		1.4
4.5-7.2 Light tan fat clay, soft, moist	163-164		49__19	3T		1.1
7.2-9.5 Dark grayish - brown lean clay, moist	165-167		56__23	3T		1.4
9.5-13.7 Light tan and gray lean clay, moist	168-170			3T		1.4
	171-172			3T		1.7
13.7-17.0 Dark grayish brown lean clay, moist	173					
	174-176			3T		1.1
17.0-22.5 Light brown lean clay, moist, medium stiff	177-179			3T		1.6
	180-182			3T		1.8
Boring completed at depth of 22.5						

Water Level: 6/2/99

Elevation: 0, -5, -10, -15, -20, -25, -30, -35
Depth: 0, 5, 10, 15, 20, 25, 30, 35

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Project Name: I-70 - Emma Site	**Project NO: Test 7**
Location: Eastbound on-ramp @ Jct. Rt. Y & I-70	**Boring Number: MU7**
Ground Elevation:	**Logged By: Hilchen**
Drilling Date: 6/1/1999	**Driller: Dodds**

Weather: Cloudy with occasional showers	Auger Method:	Driller's Hole NO: V-99-36A
Conditions:	SPT Hammer Efficiency:	Drilling Equip: CME 850 G7950

Water Level: GWT not encountered

Depth	Soil Description	Sample NO	SPT N60	Sample Type	Strength Test	Recovery (ft)
0.0-2.5	Concrete and Rubble			3T		
2.5-3.8	Light brown fat clay, moist, stiff	203,205		3T		1.9
3.8-4.5	Dark brown fat clay, moist, stiff					
4.5-5.7	Light brown lean clay, moist	204 206		3T 3T		
5.7-7.9	Dark gray lean clay, moist, hard	207-208		3T 3T		0.4
7.9-10.0	Dark grayish - brown lean clay, moist, medium stiff	209		3T		
	Boring completed at depth of 10					

Moisture Content (%) (Including LL__PL): 10 20 30 40

Undrained Strength (TSF): TV ● PP ■ 1 2 3 4 5

Elevation: 0 -5 -10 -15 -20 -25 -30 -35

Depth: 0 5 10 15 20 25 30 35

SuperLog V3.0A CivilTech Software, USA www.civiltech.com File: C:\superlog3\project\EMMA_boring.log Date: 10/16/2003

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Project Name: I-70 - Emma Site	**Project NO: Test 7**
Location: Eastbound on-ramp @ Jct. Rt. Y & I-70	**Boring Number: MU8**
Ground Elevation:	**Logged By: Miller**
Drilling Date: 6/1/1999 & 6/3/1999	**Driller: Lamberson**

Weather: Partly Cloudy, 80 deg.	Auger Method:	Driller's Hole NO: L-99-25
Conditions:	SPT Hammer Efficiency:	Drilling Equip: Failing 1500 67889

Elevation	Profile	Water Level	Depth	Soil Description	Sample NO	SPT N60	Moisture Content (%) (Including LL__PL) 10 20 30 40	Undrained Strength (TSF) TV ● PP ■ 1 2 3 4 5	Sample Type	Strength Test	Recovery (ft)
0		GWT not encountered	0	0.0-1.0 Asphalt and base rock							
				1.0-20.0 Brown and gray fat clay, trace gravel, stiff - gravel and cobbles (17.5-17.9)	130-132				3T		2.0
-5			5		133-135				3T		2.0
					136-138				3T		2.1
-10			10								
					139-140				3T		2.2
-15			15		141-142				3T		2.0
					143-145				3T		1.6
-20			20	20.0-22.5 Dark gray lean clay, very stiff	146				3T		1.7
				22.5-26.0 Dark gray lean clay with gravel, some organics, hard	147				3T		1.2
-25			25	26.0-30.0 Reddish - brown lean clay with gravel, stiff	148-150				3T		2.0
					151-152				3T		1.0
-30			30	Boring completed at depth of 30.0							
-35			35								

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Project Name: I-70 - Emma Site | **Project NO: Test 7**

Location: Eastbound on-ramp @ Jct. Rt. Y & I-70 | **Boring Number: MU9**

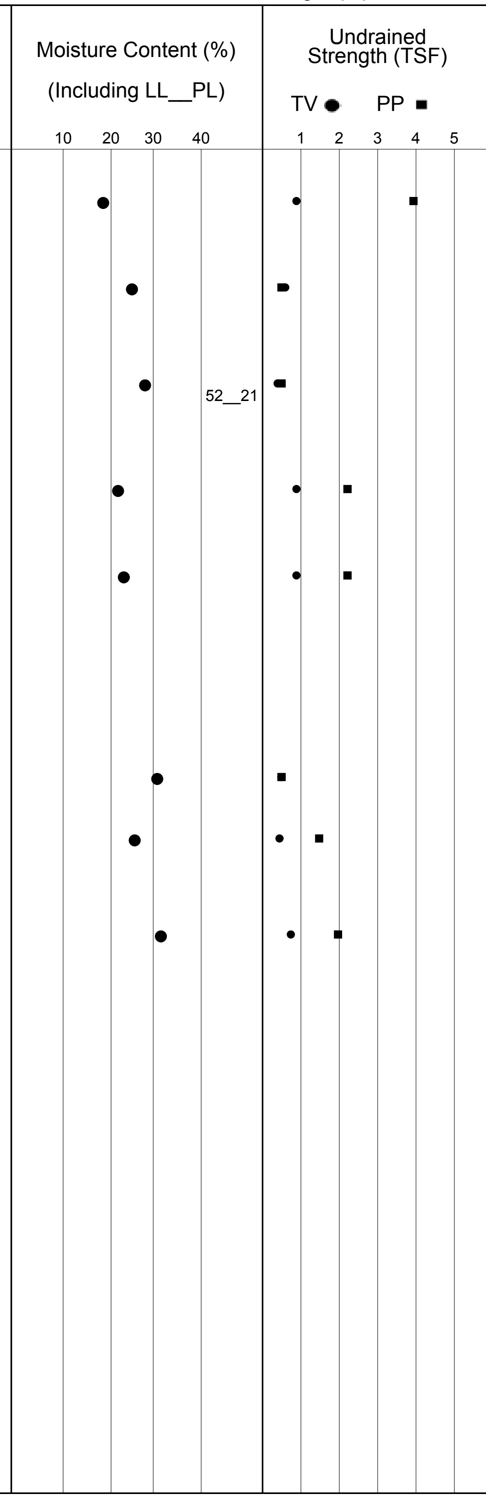
Ground Elevation: | **Logged By: Hilchen**

Drilling Date: 6/1/1999 | **Driller: Varner**

Weather: Cloudy with occasional showers | Auger Method: | Driller's Hole NO: Y-99-39

Conditions: | SPT Hammer Efficiency: | Drilling Equip: Simco 4000 G6944

Elevation	Profile	Water Level	Depth	Soil Description	Sample NO	SPT N60	Moisture Content (%) (Including LL__PL)	Undrained Strength (TSF) TV ● PP ■	Sample Type	Strength Test	Recovery (ft)
0			0	0.0-4.5 Light brown lean clay, moist, hard to medium stiff	183-184				3T		1.1
					185				3T		0.6
-5			5	4.5-7.0 Tan and gray fat clay with silt, moist, soft	186-188		52__21		3T		1.5
				7.0-12.0 Tan and gray fat clay with silt, moist, laminated layers of gray and tan fat clay	189-191				3T		1.2
-10			10		192-194				3T		1.5
		6/2/99		12.0-12.5 Dark gray lean clay with reddish - brown Mg nodules, moist							
				12.5-13.5 Cobbles and concrete					3T		
-15			15	13.5-18.2 Gray and tan fat clay, moist, medium stiff to stiff	195-197				3T		0.7
					198-200				3T		2.0
				-- shelby tube refusal @ 18.2							
-20			20		201-202				3T		0.7
				Boring completed at depth of 20.7							
-25			25								
-30			30								
-35			35								



Refusal @ 18.2 -- cleaned out to 20.0

(Sample not used 12.5-15.0)

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Project Name: I-70 - Emma Site | **Project NO: Test 7**

Location: Eastbound on-ramp @ Jct. Rt. Y & I-70 | **Boring Number: MU10**

Ground Elevation: | **Logged By: Hilchen**

Drilling Date: 6/1/1999 | **Driller: Dodds**

Weather: Cloudy with occasional showers | Auger Method: | Driller's Hole NO: V-99-36B

Conditions: | SPT Hammer Efficiency: | Drilling Equip: CME 850 G7950

Water Level: 6/2/99

Depth	Soil Description	Sample NO	SPT N60	Sample Type	Strength Test	Recovery (ft)
0.0-2.0	0.0-2.0 Light brown lean clay, moist, soft	210-212		3T		1.5
2.0-4.0	2.0-4.0 Dark brown lean clay, moist, medium stiff	213-214		3T		1.1
4.0-10.0	4.0-10.0 Light brown and gray lean clay, moist, stiff	215-217		3T		1.7
		218-219		3T		1.1
10.0	Boring completed at depth of 10.0					

Moisture Content (%) (Including LL__PL): 10 20 30 40

Undrained Strength (TSF): TV ● PP ■ 1 2 3 4 5

Elevation: 0, -5, -10, -15, -20, -25, -30, -35

Depth: 0, 5, 10, 15, 20, 25, 30, 35

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Project Name: I-70 - Emma Site | **Project NO: Test 7**

Location: Eastbound on-ramp @ Jct. Rt. Y & I-70 | **Boring Number: MU11**

Ground Elevation: | **Logged By: Hilchen**

Drilling Date: 6/2/1999 | **Driller: Dodds**

Weather: Clear & 80 deg | Auger Method: | Driller's Hole NO: V-99-36C

Conditions: | SPT Hammer Efficiency: | Drilling Equip: CME 850 G7950

Water Level: GWT not encountered

Soil Description	Sample NO	SPT N60	Moisture Content (%) (Including LL__PL)	Sample Type	Recovery (ft)
0.0-2.5 Pavement and gravel					
2.5-7.0 Light brown and gray fat clay, moist, stiff	227-229			3T	1.3
	230-231		57__20	3T	1.0
7.0-8.2 Gray and brown fat clay, moist, stiff					
8.2-13.1 Brown fat clay, moist, stiff, with reddish - brown Mg nodules	232-234			3T	1.7
	235-236			3T	1.1
	237			3T	1.4
13.1-17.0 Dark brown and gray fat clay, moist, stiff	238-239			3T	
	240-241			3T	1.3
17.0-19.5 Concrete					
19.5-25.0 Gray and tan fat clay, moist, very stiff	242-243			3T	1.1
	244-246			3T	1.2
25.0-28.0 Tan and gray weathered shale, hard, moist	247			3T	0.6
	248	13/21/24 61		SS	1.5
	249			3T	1.1
28.0-29.8 Unconsolidated mudstone to unconsolidated shale, moist, hard, with reddish - brown iron oxide mottles	250	12/23/30 72		SS	1.5
Boring completed at depth of 29.8					

SPT corrected N60 values given below blow sequence

Shelby tube refusal at 28.3' -- sample too disturbed for use

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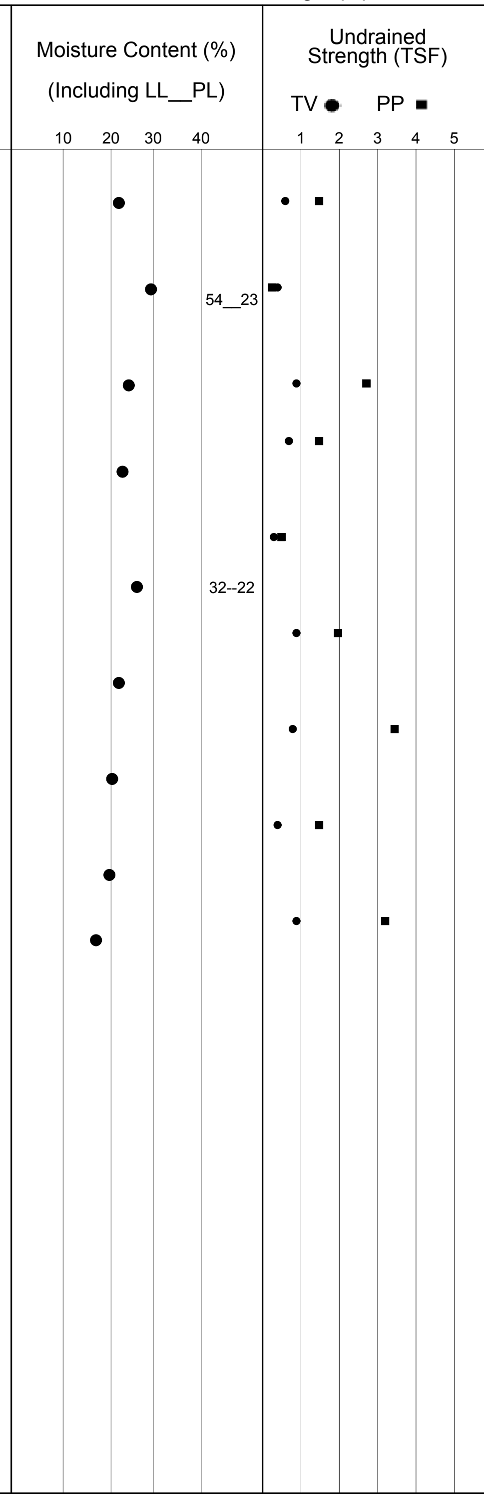
Project Name: I-70 - Emma Site	**Project NO: Test 7**
Location: Eastbound on-ramp @ Jct. Rt. Y & I-70	**Boring Number: MU12**
Ground Elevation:	**Logged By: Hilchen**
Drilling Date: 6/2/1999	**Driller: Varner**

Weather: Clear 80's	Auger Method:	Driller's Hole NO: Y-99-40
Conditions:	SPT Hammer Efficiency:	Drilling Equip: Simco 4000 TR-2 G6944

Water Level: GWT not encountered

Depth	Soil Description
0.0-4.5	Light brown and gray fat clay, moist, soft to medium stiff
4.5-7.0	Dark brown fat clay, moist, stiff
7.0-9.0	Light brown to tan lean clay, moist
9.0-14.0	Dark brown lean clay, moist, medium stiff to stiff
14.0-20.9	Light tan gray fat clay, moist, stiff, with slickensides
	Boring completed at depth of 20.9

Sample NO	SPT N60	Including LL__PL	Sample Type	Strength Test	Recovery (ft)
273			3T		0.6
274-276		54__23	3T	CU(2)	1.3
277-278			3T	CU CU	1.3
279					
280-281			3T		0.7
282-284		32--22	3T	CD(2)	1.6
285-287			3T	CU CU(2)	1.2
288-289			3T	CD	0.8
290-291			3T		0.9
292-293			3T		0.9



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Project Name: I-70 - Emma Site	**Project NO: Test 7**
Location: Eastbound on-ramp @ Jct. Rt. Y & I-70	**Boring Number: MU13**
Ground Elevation:	**Logged By: Hilchen**
Drilling Date: 6/1/1999	**Driller: Dodds**

Weather: Cloudy with occasional showers	Auger Method:	Driller's Hole NO: V-99-36C
Conditions:	SPT Hammer Efficiency:	Drilling Equip: CME 850 G7950

Elevation	Profile	Water Level	Depth	Soil Description	Sample NO	SPT N60	Moisture Content (%) (Including LL__PL) 10 20 30 40	Undrained Strength (TSF) TV ● PP ■ 1 2 3 4 5	Sample Type	Strength Test	Recovery (ft)
0		GWT not encountered	0	0.0-2.5 Rubble and concrete, free water at 1.5'					3T		
				2.5-6.1 Brown and reddish brown lean clay, very stiff	220-221				3T		2.5
-5			5	6.1-9.5 Greenish gray fat clay, moist	222-224				3T		1.4
					225-226				3T		1.2
-10			10	Boring completed at depth of 9.5							
-15			15								
-20			20								
-25			25								
-30			30								
-35			35								

(0.0-2.5) No recovery -- rock, gravel, and concrete

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Project Name: I-70 - Emma Site | **Project NO: Test 7**

Location: Eastbound on-ramp @ Jct. Rt. Y & I-70 | **Boring Number: MU14**

Ground Elevation: | **Logged By: Hilchen**

Drilling Date: 6/2/1999 | **Driller: Dodds**

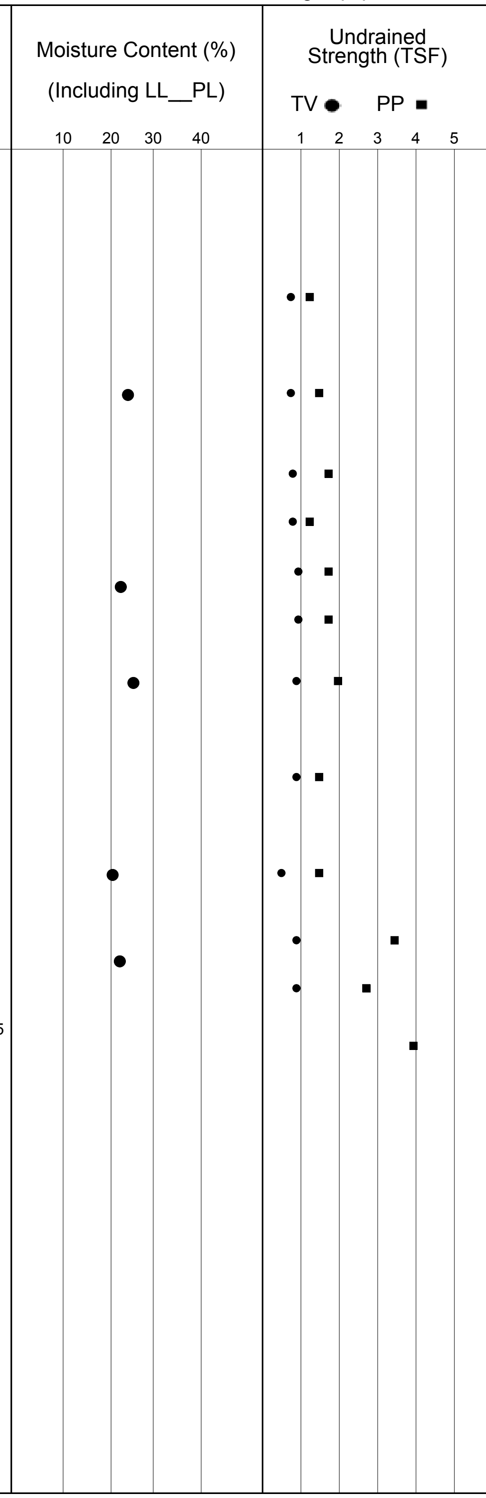
Weather: Clear & Sunny, breezy, mid 80's | Auger Method: | Driller's Hole NO: V-99-37

Conditions: | SPT Hammer Efficiency: | Drilling Equip: CME 850 G7950

Water Level: GWT not encountered

Soil Description
0.0-1.3 Asphalt & base rock
1.3-9.2 Brown & gray fat clay, moist, very stiff
9.2-11.4 Brown fat clay, moist, stiff
11.4-11.8 Tan & brown fat clay, moist, stiff, with Mg nodules
11.8-16.5 Brown & gray fat clay, moist
16.5-17.1 Concrete & gravel
17.1-18.5 Greenish - dark brown lean clay with silt, moist, soft
18.5-21.0 Light brown & gray fat clay, moist, medium stiff
21.0-24.0 Gray with tan fat clay, moist, very stiff to very hard
Boring completed at depth of 24.0

Sample NO	SPT N60	Sample Type	Strength Test	Recovery (ft)
251-252		3T		1.6
253-254		3T		0.8
255-257		3T	DS	2.5
258-259		3T		2.2
260				
261-263		3T		1.8
264-266		3T		1.5
267-268		3T		2.0
269		3T		2.0
270-271				
272	8/11/15	SS		1.5



0.0-2.5 --- too much pavement & gravel to recover a sample

SPT corrected N60 value not given, just the blow sequence

DS--Direct Shear Test

University of Missouri - Columbia

Civil & EnvironmentalEngineering Department -Geotechnical Group

Project Name: I-70 - Emma Site
Project NO: Test 7
Location: Eastbound on-ramp @ Jct. Rt. Y & I-70
Boring Number: MU15
Ground Elevation:
Logged By: Miller
Drilling Date: 6/2/1999
Driller: Lamberson

Weather: Sunny, 80's
Auger Method:
Driller's Hole NO: L-99-27
Conditions:
SPT Hammer Efficiency:
Drilling Equip: Failing 1500 G7889

Depth (ft)	Soil Description
0.0-1.4	Asphalt & base rock
1.4-18.5	Gray & brown fat clay, trace fine gravel, stiff -- scattered gravel beginning @ 13.0
18.5-22.5	Dark brown lean clay, scattered gravel layers, medium stiff
22.5-25.0	Reddish - brown lean clay, trace fine gravel, hard
25.0-27.5	Gray shaley clay, scattered gravel, hard
27.5-28.5	Red weathered sandstone, hard
	Boring completed at depth of 28.5

Water Level: 6/3/99

Sample NO	SPT N60	Moisture Content (%) (Including LL__PL)	Sample Type	Strength Test	Recovery (ft)
210-212			3T		2.5
213-215		56__23	3T		2.3
216-218			3T		2.1
219-221			3T		2.1
222-224			3T		1.6
225-227			3T		1.6
228			3T		1.0
229	1/1/2		SS		1.5
230			3T		0.5
231-232			3T		1.2
233	10/18/27		SS		1.5

Undrained Strength (TSF): TV ● PP ■

3T refusal @ 18.8 -- cleaned out to 20.0'
3T refusal @ 27.0

University of Missouri - Columbia

Civil & EnvironmentalEngineering Department -Geotechnical Group

Project Name: I-70 - Emma Site | **Project NO: Test 7**

Location: Eastbound on-ramp @ Jct. Rt. Y & I-70 | **Boring Number: MU16**

Ground Elevation: | **Logged By: Hilchen**

Drilling Date: 6/2/1999 | **Driller: Varner**

Weather: Clear & Sunny, mid 80's | Auger Method: | Driller's Hole NO: Y-99-42

Conditions: | SPT Hammer Efficiency: | Drilling Equip: Simco 4000 TR-2 G6944

Elevation	Profile	Water Level	Depth	Soil Description	Sample NO	SPT N60	Moisture Content (%) (Including LL__PL)	Undrained Strength (TSF) TV ● PP ■	Sample Type	Strength Test	Recovery (ft)
0		GWT not encountered	0	0.0-7.4 Light brown lean clay with silt, moist, medium stiff	309-311				3T		1.5
					312-313		49__21		3T	DS	1.2
-5			5		314-315				3T		1.3
				7.4-8.0 Concrete rubble (fill)							
				8.0-14.0 Greenish - brown and light brown fat clay with silt, very stiff, moist							
-10			10		316-317				3T		0.9
					318-319				3T		0.9
-15			15	14.0-18.4 Unconsolidated mudstone or shale, gray to light tan, moist to dry, hard to very hard	320				3T		0.6
					321				3T		0.9
				Boring completed at depth of 18.4							
-20			20								
-25			25								
-30			30								
-35			35								

Moisture Content scale: 10 20 30 40; Undrained Strength scale: 1 2 3 4 5

3T refusal @ 14.0' -- cleaned out to 15.0'

3T refusal @ 15.8' -- cleaned out to 17.5'

DS--Direct Shear Test

University of Missouri - Columbia
Civil & EnvironmentalEngineering Department -Geotechnical Group

Project Name: I-70 - Emma Site
Location: Eastbound on-ramp @ Jct. Rt. Y & I-70
Ground Elevation:
Drilling Date: 6/2/1999

Project NO: Test 7
Boring Number: MU17
Logged By: Hilchen
Driller: Varner

Weather: Clear, Sunny
Conditions:
Auger Method:
SPT Hammer Efficiency:
Driller's Hole NO: Y-99-41
Drilling Equip: Simco 4000 TR-2 G6944

Water Level: GWT not encountered

Depth (ft)	Soil Description
0.0-7.0	Light brown and gray fat clay, moist, stiff, with reddish - brown Mg nodules
7.0-7.5	Concrete rubble
7.5-9.5	Dark greenish - brown lean clay with silt, moist, with brown Mg nodules, stiff, free water @ 7.5'
9.5-15.0	Brown to tan fat clay, moist, very stiff
15.0-18.4	Gray unconsolidated siltstone or mudstone
	Boring completed at depth of 18.4

Sample NO	SPT N60	Moisture Content (%) (Including LL__PL)	Sample Type	Strength Test	Recovery (ft)
294			3T		0.5
295-296		50__22	3T		0.9
297-299			3T		1.5
300-301			3T		1.2
302-303			3T		1.2
304-305			3T		1.6
306			3T		0.5
307-308			3T		0.9

Undrained Strength (TSF): TV ● PP ■

3T refusal @ 14.5' -- cleaned out to 15.0'
3T refusal @ 16.0' -- cleaned out to 17.5'

Appendix D. Boring Logs for US54-Fulton Site

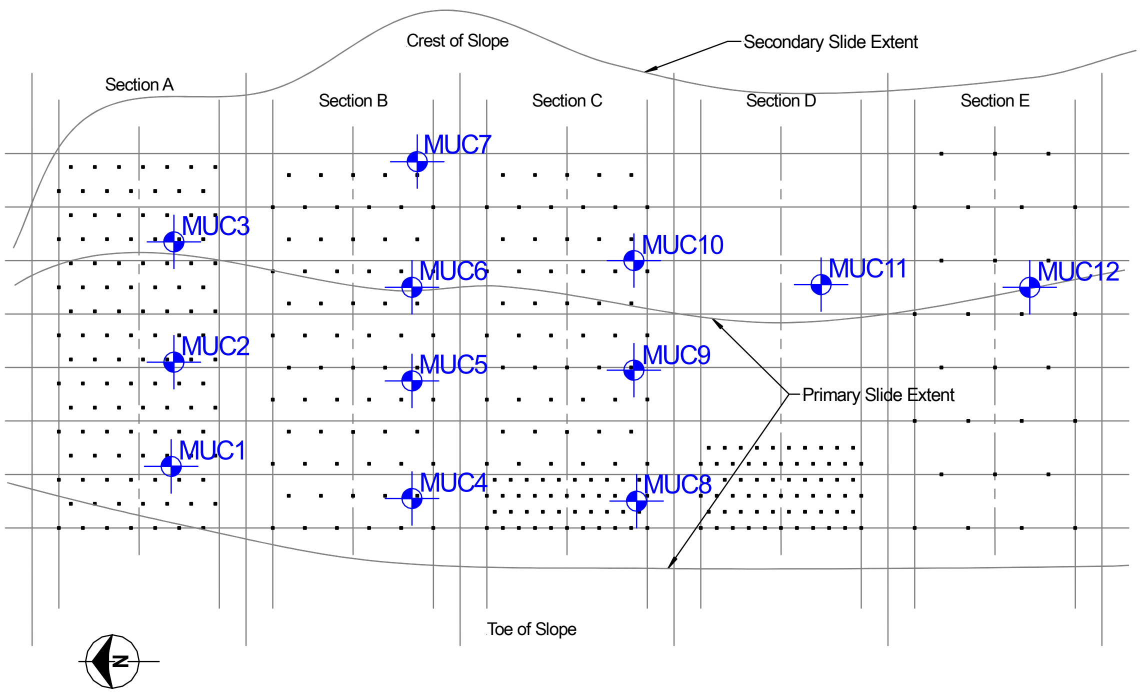


Figure D.1 Plan view of US54-Fulton site showing approximate boring locations.

University of Missouri - Columbia

Civil & EnvironmentalEngineering Department -Geotechnical Group

Project Name: US 54 - Fulton Site | **Project NO: 906 RDT RI98-701**

Location: 500 ft South of Richland Creek | **Boring Number: MUC-1**

Ground Elevation: | **Logged By: Fennessey**

Drilling Date: 9/25/2000 | **Driller: Hess**

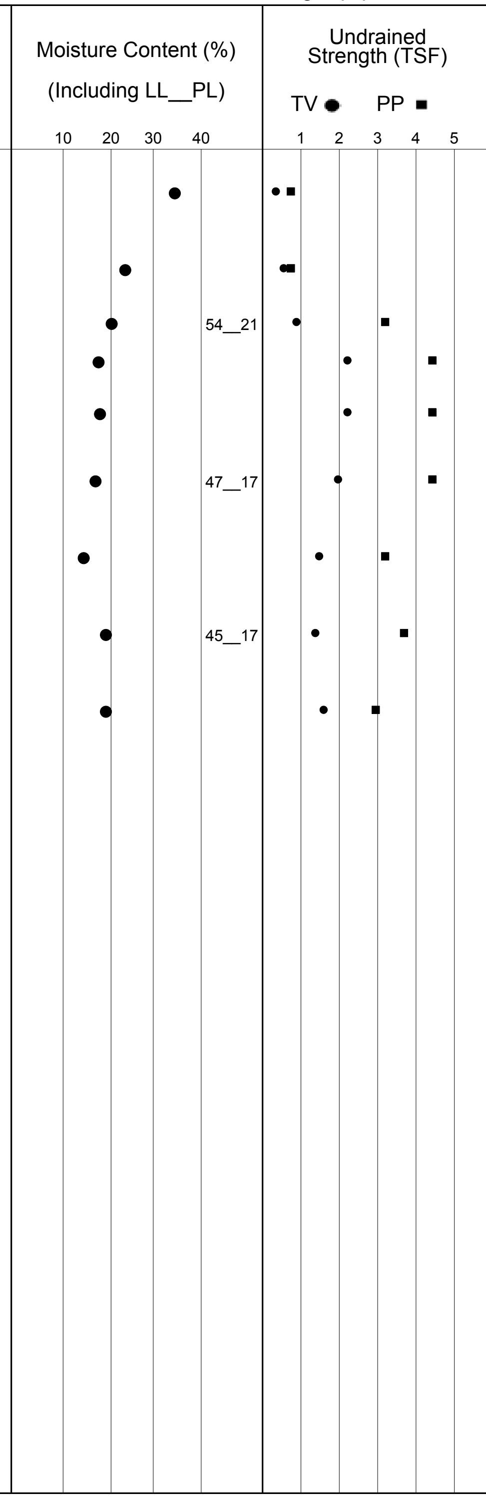
Weather: Overcast, Mild | Auger Method: 4" Hollow Stem | Driller's Hole NO: Y-00-106

Conditions: | SPT Hammer Efficiency: | Drilling Equip: Versa Drill G8641

Water Level: GWT not encountered

Depth	Soil Description	Sample NO	SPT N60	LL__PL	Sample Type	Strength Test	Recovery (ft)
0	--0.0-2.0 Gray - brown lean clay, medium stiff, moist (roots & organics to 0.4)	146-147			AL-3T	CU	1.1
	--2.0-4.8 Yellow - brown to light gray mottled, medium stiff, moist, trace gravel	148-149			3T-AL	CU(2)	1.1
5		150,152		54__21	3T-AL		0.8
	--4.8-9.5 Tan fat clay, hard, moist, trace sand and gravel (possible till)	151,153			3T-AL		1.0
		154-155			3T-AL		1.4
		156-158		47__17	3T*-3T*-AL		1.9
10	--9.5-15.5 Gray - brown to brown lean clay, very stiff, moist, trace sand and gravel (possible till)	159			AL-**		1.4
		160-162		45__17	3T-3T-AL		2.0
15		163-165			3T-3T-AL		1.6
	Boring completed at depth of 15.5						

Elevation / Depth scale: 0 to -35 / 0 to 35 ft



* Partially disturbed (creased on side) ** damaged tube

3T--3" dia. shelby tube AL--bag sample for Atterberg Limits

Note: Shelby Tube refusal at 7.5 ft.

University of Missouri - Columbia

Civil & EnvironmentalEngineering Department -Geotechnical Group

Project Name: US 54 - Fulton Site | **Project NO: 906 RDT RI98-701**

Location: 500 ft South of Richland Creek | **Boring Number: MUC-2**

Ground Elevation: | **Logged By: Fennessey/Parra**

Drilling Date: 9/25/2000 - 9/26/2000 | **Driller: Hess**

Weather: | Auger Method: 4" Hollow Stem | Driller's Hole NO: Y-00-107

Conditions: | SPT Hammer Efficiency: 75 | Drilling Equip: Versa Drill G8641

Depth	Soil Description	Sample NO	SPT N60	Moisture Content (%) (Including LL__PL)	Undrained Strength (TSF) PP	Sample Type	Strength Test	Recovery (ft)
0	--0.0-3.0 Brown lean clay, scattered snad, dry to moist	166	1/1/ 1			SS		0.8
		167	2/3/4 9	38__18	■	SS		1.4
	--3.0-4.1 Gray - brown lean clay, medium stiff to stiff, moist, trace sand	168	3/3/4 9		■	SS		1.35
5	--4.1-6.5 Gray lean clay, medium stiff to stiff, moist, trace sand	169	3/3/4 9	39__21	■	SS		1.5
	--6.5-9.5 Gray - brown to dark yellow - brown lean clay, medium stiff to stiff, moist, trace sand	170	2/4/4 10		■	SS		.75
		171	3/4/7 14		■	SS		.95
10	--9.5-20.5 Olive and light gray lean clay, very stiff to hard, moist, trace to scattered sand and gravel	172	5/5/8 16	50__18	■	SS		1.5
		173	6/7/11 23		■	SS		1.5
		174	5/10/11 26		■	SS		1.5
15		175	5/8/10 23	47__17	■	SS		1.5
		176	8/9/9 23		■	SS		1.2
		177	5/7/11 23		■	SS		1.5
20		178	5/8/9 21	45__17	■	SS		1.5
	Boring completed at depth of 20.5							

Elevation scale: 0, -5, -10, -15, -20, -25, -30, -35. Depth scale: 0, 5, 10, 15, 20, 25, 30, 35. Water Level: GWT not encountered. Moisture Content axis: 10 20 30 40. Undrained Strength axis: 1 2 3 4 5; TV ● PP ■

3T--3" dia. shelby tube AL--bag sample for Atterberg Limits

Note: SPT corrected N60 values given below blow sequence

--Cleaned out at 16 ft and stopped for day -- restarted 8:30 am 9/26

SuperLog V3.0A CivilTech Software, USA www.civiltech.com File: C:\superlog3\project\US-54_boring.log Date: 10/16/2003

University of Missouri - Columbia

Civil & EnvironmentalEngineering Department -Geotechnical Group

Project Name: US 54 - Fulton Site	**Project NO: 906 RDT RI98-701**
Location: 500 ft South of Richland Creek	**Boring Number: MUC-3**
Ground Elevation:	**Logged By: Fennessey**
Drilling Date: 9/26/2000	**Driller: Hess**

Weather: Clear, Warm	Auger Method: 4" Hollow Stem	Driller's Hole NO: Y-00-108
Conditions:	SPT Hammer Efficiency:	Drilling Equip: Versa Drill G8641

Water Level: GWT not encountered

Depth	Soil Description
0.0-0.7	--0.0-0.7 Brown to tan lean clay, medium stiff, dry, with root and organics
0.7-3.6	--0.7-3.6 Gray - brown lean clay, dry, hard, trace gravel & sand
3.6-12.0	--3.6-12.0 Gray - brown with tan & black mottled lean clay, stiff to very stiff, moist with trace sand and gravel
12.0-13.7	--12.0-13.7 Gray lean clay, very stiff, moist, trace sand and gravel
13.7-21.3	--13.7-21.3 Olive - brown with brown & gray mottled lean clay, very stiff to hard, moist, trace sand
	Boring completed at depth of 21.3

Sample NO	SPT N60	Moisture Content (%) (Including LL__PL)	Sample Type	Strength Test	Recovery (ft)
			**		
179			AL		1.8
180		36__15	**-AL		1.5
181-182		37__15	3T-AL		1.2
183-184			3T-AL		0.8
185-186		39__16	3T-AL	CU(2)	0.9
187-188			3T-AL		1.0
189-190		40__13	3T-AL		1.3
191			AL		
192-193		62__17	3T-AL		1.5
194-195			3T-AL		1.6
196-197			3T-AL		1.0
198-199		48__15	3T-AL		0.8
200-201			3T-AL		1.7

Undrained Strength (TSF): TV ● PP ■

** sample grooved by rock, did not keep 3T sample

3T--3" dia. shelby tube AL--bag sample for Atterberg Limits

3T refusal at 17.4, 18.5, 19.5, 21.3 ft.

University of Missouri - Columbia

Civil & EnvironmentalEngineering Department -Geotechnical Group

Project Name: US 54 - Fulton Site	**Project NO: 906 RDT RI98-701**
Location: 500 ft South of Richland Creek	**Boring Number: MUC-4**
Ground Elevation:	**Logged By: Less**
Drilling Date: 10/2/2000	**Driller: Hees**

Weather: Sunny, Mild	Auger Method: 4" Hollow Stem	Driller's Hole NO: Y-00-113
Conditions:	SPT Hammer Efficiency:	Drilling Equip: Versa Drill 4000 TR-2

Water Level: 10/2/00 (at approx. 7 ft depth)

Depth (ft)	Soil Description	Sample NO	SPT N60	Sample Type	Strength Test	Recovery (ft)
0.0-0.9	--0.0-0.9 Brown to tan lean clay	401		AL	CU(2)	1.7
0.9-7.6	--0.9-7.6 Brown lean clay, medium stiff, trace gravel, moist gray mottling @ 5.5 ft.	400,402		AL-3T		1.7
		404-405		AL-3T	CU	1.3
		406-407		AL-3T	CU(2)	2.0
		408-409		AL-3T		1.5
7.6-11.5	--7.6-11.5 Brown fat clay, with gravel, stiff to very stiff, moist with gray mottles	410-411		AL-3T		1.5
		412-413		AL-3T		1.8
11.5-16.0	--11.5-16.0 Brown lean clay with gravel, stiff to very stiff, moist, abundant gravel 11.7 to 12.5 (too heavy to sample), trace black and gray mottles	414		AL		0.4
		415		AL		
		416-417		AL-3T		1.6
		418-419		AL-3T		1.5
16.0	Boring completed at depth of 16.0					

Columns also include: Elevation (0 to -35), Profile, Moisture Content (%) (Including LL__PL) (10, 20, 30, 40), Undrained Strength (TSF) TV ● PP ■ (1, 2, 3, 4, 5).

3T--3" dia. shelby tube AL--bag sample for Atterberg Limits

Refusal at 11.7, cleaned out to 12.5

Sample damaged (14.5-16.0)

University of Missouri - Columbia

Civil & EnvironmentalEngineering Department -Geotechnical Group

Project Name: US 54 - Fulton Site | **Project NO: 906 RDT RI98-701**

Location: 500 ft South of Richland Creek | **Boring Number: MUC-5**

Ground Elevation: | **Logged By: Fennessey**

Drilling Date: 9/28/2000 | **Driller: Hees**

Weather: Clear, Warm | Auger Method: 4" Hollow Stem | Driller's Hole NO: Y-00-112

Conditions: | SPT Hammer Efficiency: | Drilling Equip: Versa Drill G8641

Elevation	Depth	Soil Description	Sample NO	SPT N60	Moisture Content (%) (Including LL__PL)	Sample Type	Strength Test	Recovery (ft)
0	0	--0.0-3.5 Tan and brown lean clay, very stiff, moist, --brown and dry to 0.3 ft. w/ organics	272-273		45__14	3T-AL		1.2
			275			3T		0.9
		--3.5-9.3 Gray - brown with tan & gray mottles, lean clay, trace sand, very stiff and moist, --hard w/ brown mottles below 6.0	276			AL		
-5	5		277-278		40__15	3T-AL		0.9
			279-280			3T-AL		1.4
			281-282		48__17	3T-AL		1.1
-10	10	--9.3-14.0 Gray lean clay, trace sand and gravel, very stiff, moist	283			AL		
			284-286		41__10	3T-3T-AL		1.6
			287			AL-**		1.3
-15	15	--14.0-19.9 Olive - gray & gray fat clay, trace to scattered gravel & sand, very stiff to hard, moist, --limestone and chert gravel @ 15.8, --gravelly 16.6-17.3	288-289			3T-AL		1.5
			290		53__16	AL		0.4
			291			AL-**		0.6
			292			AL-**		0.8
			293	9/12/14 33	52__16	SS		1.5
-20	20	Boring completed at depth of 19.9						
-25	25							
-30	30							
-35	35							

Water Level: 10/2/00

Undrained Strength (TSF): TV ● PP ■ (1, 2, 3, 4, 5)

Moisture Content scale: 10, 20, 30, 40

3T--3" dia. shelby tube AL--bag sample for Atterberg Limits

Refusal at 15.5, 15.8, 16.6

** Shelby tube crushed by cobble (too disturbed for 3T sample)

Note: SPT corrected N60 values given below blow sequence

University of Missouri - Columbia

Civil & EnvironmentalEngineering Department -Geotechnical Group

Project Name: US 54 - Fulton Site | **Project NO: 906 RDT RI98-701**

Location: 500 ft South of Richland Creek | **Boring Number: MUC-6**

Ground Elevation: | **Logged By: Fennessey**

Drilling Date: 9/26/2000 - 9/27/2000 | **Driller: Hees**

Weather: Clear & Warm both days | Auger Method: 4" Hollow Stem | Driller's Hole NO: Y-00-109

Conditions: | SPT Hammer Efficiency: | Drilling Equip: Versa Drill G8641

Elevation / Profile / Water Level / Depth

Water Level: 10/2/00 (at approx. 8.5 ft); 9/28/00 (at approx. 17 ft)

Soil Description

--0.0-0.4 Brown lean clay, medium stiff to stiff

--0.4-1.0 Tan silty fine san, med dense, dry

--1.0-3.0 Gray - brown with tan & gray mottles, very hard, dry

--3.0-6.0 Tan silty fine sand, dense, dry, --gray 3.0-3.1, --trace lean clay & lean clay pockets with depth

--6.0-7.5 Tan and gray mottled lean clay, trace sand, hard, moist

--7.5-16.9 Gray - brown lean clay, trace sand, very stiff, moist, --@ 7.5ft. 3T pushed 0.5 ft. under weight of drill head, --wet!! @ 15.5, --becoming gray @ 16.6

--16.9-19.6 Olive - gray with tan & gray mottle, fat clay, hard, moist, trace sand & gravel

Boring completed at depth of 19.6

Sample NO	SPT N60	Moisture Content (%) (Including LL__PL)	Undrained Strength (TSF) TV ● PP ■	Sample Type	Strength Test	Recovery (ft)
202-203				AL-AL		1.2
204		30__16		AL		
205				AL		1.8
206				AL		0.5
207		45__15		SS		1.1
	3/4/4					
208-209				3T-AL		1.3
210-211				3T-AL		1.0
212-213		43__16		3T-AL		1.7
214-215				3T-AL		1.3
216-217				3T-AL		1.3
218				AL		
219		53__17		AL		0.5
220				AL-**		0.7
221				AL-**		0.6

3T--3" dia. shelby tube AL--bag sample for Atterberg Limits

SS-- Split Spoon sampler Stopped for day @ 11.5 ft.

--3T Refusal @ 18.0 & 19.6 (3T Refusal @ 18.7 drilled out to19.0)

** Sample too disturbed for 3T

University of Missouri - Columbia

Civil & EnvironmentalEngineering Department -Geotechnical Group

Project Name: US 54 - Fulton Site | **Project NO: 906 RDT RI98-701**

Location: 500 ft South of Richland Creek | **Boring Number: MUC-7**

Ground Elevation: | **Logged By: Fennessey**

Drilling Date: 9/27/2000 | **Driller: Hees**

Weather: Clear, Warm | Auger Method: 4" Hollow Stem | Driller's Hole NO: Y-00-110

Conditions: | SPT Hammer Efficiency: | Drilling Equip: Versa Drill G8641

Water Level: GWT not encountered

Soil Description

- --0.0-7.5 Brown to tan lean clay, very hard, dry, trace to scattered gravel, --brown moist w/ organics to 0.3, --lense of crystallization (secondary depostion) @6.2 ft.
- --7.5-10.3 Gray - brown to tan lean clay, hard, moist, trace sand
- --10.3-12.4 Gray lean clay, hard moist, trace sand
- --12.4-15.0 Gray - brown lean clay, trace sand, hard, moist, --lense of crystallization (secondary deposition @ 50 deg from hor in sample)
- --15.0-15.3 Tan silt, hard, dry
- --15.3-16.5 Gray - brown lean clay, trace sand, hard, moist to 16.0 ft, becoming silty 16.0-16.5
- --16.5-17.2 Gray lean clay, trace sand, very stiff to hard, moist
- --17.2-18.3 Gray - brown w/ brown & tan mottled lean clay, trace sand, hard, moist
- --18.3-19.5 Gray lean clay, trace sand, very stiff, moist
- --19.5-22.0 Gray - brown lean clay, hard, moist, trace sand
- --22.0-26.1 Gray lean clay, trace sand, very stiff to hard, moist, trace gravel, --limestone cobble/boulder @ 25.0 ft., clean out to 26.5 ft.
- --27.1-27.8 Olive - gray fat clay, very stiff, moist
- --27.8-28.3 Gray lean clay, trace sand, hard, moist
- --28.3-30.5 Olive - gray fat clay w/ tan mottles, trace gravel, hard, moist

Sample NO	SPT N60	Moisture Content (%) (Including LL__PL)	Sample Type	Recovery (ft)
222-223			3T-AL	1.0
224			AL	0.3
225	7/9/9 23		SS	1.2
226	8/4/8 15		SS	1.2
227		37__16	AL-*	1.9
228-229			3T-AL	1.0
230-231			3T-AL	2.0
232-233			AL	
234			3T	1.4
235			3T	
236		37__16	AL-**	0.7
237			AL	0.8
238			AL	
239			AL	1.9
240			3T	
241			AL	
242			AL	1.5
243			3T	
244			AL	
245-246			3T	1.3
247-248			AL	1.3
249			3T	1.2
250	100+		SS	0.1
251			AL	2.1
252			3T	
253			AL	1.9
254-255			3T-AL	

Undrained Strength (TSF): TV ● PP ■

* sample too dry and broken for 3T

** collapsed by gravel, no undist sample, poor quality

3T--3" dia. shelby tube AL--bag sample for Atterberg Limits

SS--Split Spoon

University of Missouri - Columbia

Civil & EnvironmentalEngineering Department -Geotechnical Group

Project Name: US 54 - Fulton Site
Location: 500 ft South of Richland Creek
Ground Elevation:
Drilling Date: 10-2-2000

Project NO: 906 RDT RI98-701
Boring Number: MUC-8
Logged By: Less
Driller: Hees

Weather: Sunny, Warm
Conditions:

Auger Method: 4" Hollow Stem
SPT Hammer Efficiency:

Driller's Hole NO: Y-00-114
Drilling Equip: Versa Drill 4000TR-2

Elevation	Depth	Water Level	Soil Description	Sample NO	SPT N60	Sample Type	Strength Test	Recovery (ft)
0	0		--0.0-0.6 Brown lean clay, roots, medium stiff, dry	420-421		AL-AL		1.9
			--0.6-6.6 Brown lean clay, medium stiff, moist, gray mottles with gravel	422-423		AL-3T		1.5
-5	5			424-425		AL-3T	CU	1.9
			--6.6-9.5 Gray lean clay, brown mottles, stiff to very stiff, moist	426-427		AL-3T		1.8
				428-429		AL-3T		2.0
-10	10	10/2/00	--9.5-15.0 Brown lean to fat clay, trace sand, medium stiff, moist, --WET at 10.8 ft.	430-431		AL-3T		1.5
				432-433		AL-3T		1.0
				434		AL-**		1.5
-15	15		Boring completed at depth of 15.0					
-20	20							
-25	25							
-30	30							
-35	35							

Moisture Content (%) (Including LL__PL): 10 20 30 40

Undrained Strength (TSF): TV ● PP ■ 1 2 3 4 5

3T--3" dia. shelby tube AL--bag sample for Atterberg Limits
Refusal at 11.5, cleaned out to 12.0
** final 3T sample damaged (13.0-15.0) did not keep

University of Missouri - Columbia

Civil & EnvironmentalEngineering Department -Geotechnical Group

Project Name: US 54 - Fulton Site	**Project NO: 906 RDT RI98-701**
Location: 500 ft South of Richland Creek	**Boring Number: MUC-9**
Ground Elevation:	**Logged By: Fennessey**
Drilling Date: 9/27/2000 - 9/28/2000	**Driller: Hees**

Weather:	Auger Method: 4" Hollow Stem	Driller's Hole NO: Y-00-111
Conditions:	SPT Hammer Efficiency: 75	Drilling Equip: Versa Drill G8641

Water Level: GWT not encountered

Soil Description

- --0.0-3.0 Light brown to tan lean clay, trace sand, hard to very stiff, moist, --brown with organics to 0.2
- --3.0-8.0 Gray - Brown with tan & brown mottled lean clay, trace sand, very stiff, moist
- --8.0-9.6 Gray lean clay, trace sand, very stiff, moist
- --9.6-10.5 Gray - brown lean clay, trace sand, very stiff, moist
- --10.5-13.0 Gray lean clay, trace sand, very stiff to hard, moist
- --13.0-13.5 Gray - brown with tan mottled lean clay, trace sand & gravel, very stiff, moist
- --13.5-21.0 Tan & gray - brown fat clay, trace gravel, stiff to hard, moist, --with brown mottles & trace sand and gravel below 19.4

Boring completed at depth of 21.0

Sample NO	SPT N60	Sample Type	Recovery (ft)
256	1/2/3 6	SS	1.2
257	2/3/3 8	SS	1.4
258	3/3/6 11	SS	1.4
259	3/5/7 15	SS	1.2
260	4/6/12 23	SS	1.0
261	5/8/8 20	SS	0.2
262	4/5/8 16	SS	1.4
263-264	4/6/7 16	SS	0.4
265	3/4/6 13	SS	1.6
266			
267	2/3/5 10	SS	1.6
268	3/4/7 14	SS	1.5
269	3/7/10 21	SS	1.6
270	5/5/9 18	SS	1.5
271	5/9/12 26	SS	1.5

Columns: Elevation, Profile, Water Level, Depth, Soil Description, Sample NO, SPT N60, Moisture Content (%) (Including LL__PL) 10 20 30 40, Undrained Strength (TSF) TV ● PP ■ 1 2 3 4 5, Sample Type, Strength Test, Recovery (ft)

SS- split spoon sampler

Note: SPT corrected N60 values given below blow sequence

Stopped for day @ 10.5 ft.

University of Missouri - Columbia

Civil & EnvironmentalEngineering Department -Geotechnical Group

Project Name: US 54 - Fulton Site | **Project NO: 906 RDT RI98-701**

Location: 500 ft South of Richland Creek | **Boring Number: MUC-10**

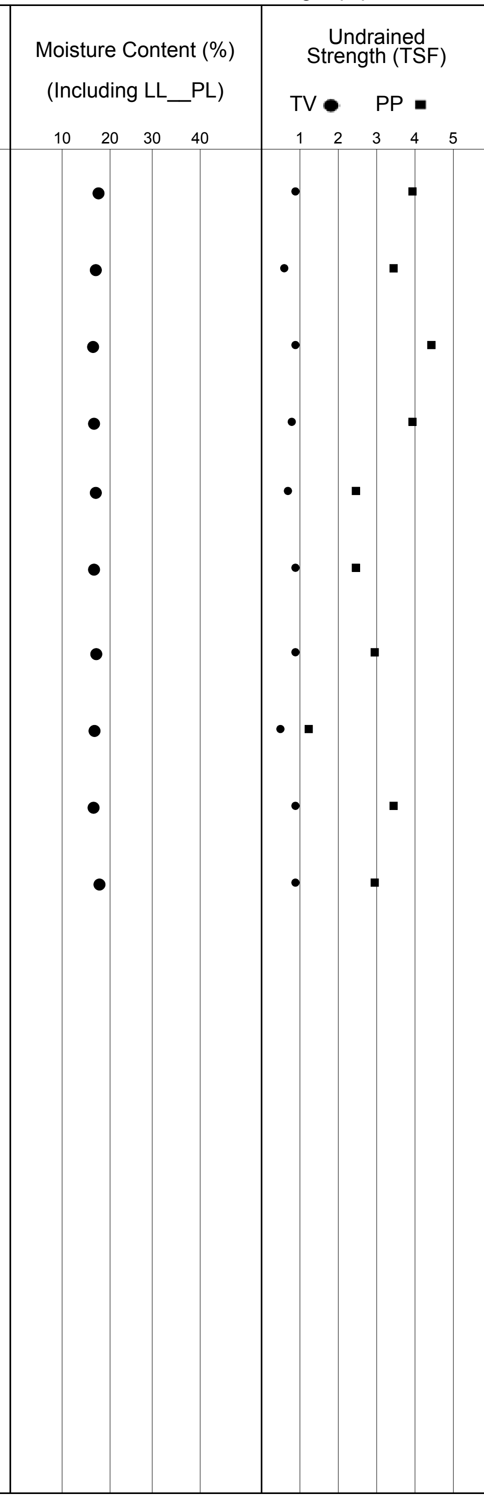
Ground Elevation: | **Logged By: Newton**

Drilling Date: 10-10-2000 | **Driller: Barnett**

Weather: Cool & Clear (50 F) | Auger Method: 4" Hollow Stem | Driller's Hole NO: 0-00-59

Conditions: | SPT Hammer Efficiency: | Drilling Equip: Versa Drill 4000TR-2

Elevation	Profile	Water Level	Depth	Soil Description	Sample NO	SPT N60	Sample Type	Strength Test	Recovery (ft)
0		GWT not encountered	0	--0.0-0.4 Black lean clay, medium stiff					
				--0.4-12.2 Brown lean clay with gray and black mottles, scattered gravel, very stiff to hard, --gravelly 9.6-10.0	621		3T		1.1
					620,622		AL,SS		1.3
-5			5		623-624		3T-AL		1.1
					625		AL		1.0
					626-627		3T-AL		1.6
-10			10		628-629		3T-AL		1.5
				--12.2-14.2 Yellowish brown mottled lean clay with scattered gravel, very stiff, gypsum crystals present @12.0	630-631		3T-AL		1.6
-15			15	--14.2-16.2 Dark brown lean clay with black mottles, stiff, scattered gravel and fine sand present, --sandy brown @ 16.0	632,637		3T-3T		1.5
				--16.2-20.0 Brown gravelly lean clay with lignite and reddish brown sandy seams, --gravelly @18	633-634		3T-AL		1.4
					635-636		AL-3T		2.1
-20			20	Boring completed at depth of 20.0					
-25			25						
-30			30						
-35			35						



AL--bag sample for Atterberg Limits

3T--3" diameter Shelby Tube

SS--Split Spoon Sampler

University of Missouri - Columbia

Civil & EnvironmentalEngineering Department -Geotechnical Group

Project Name: US 54 - Fulton Site	**Project NO: 906 RDT RI98-701**
Location: 500 ft South of Richland Creek	**Boring Number: MUC-11**
Ground Elevation:	**Logged By: Newton**
Drilling Date: 10/10/2000	**Driller: Barnett**

Weather:	Auger Method: 4" Hollow Stem	Driller's Hole NO: 0-00-60
Conditions:	SPT Hammer Efficiency:	Drilling Equip: Versa Drill G7211

Water Level: GWT not encountered

Depth	Soil Description	Sample NO	SPT N60	Sample Type
0.0-4.8	--0.0-4.8 Dark brown and gray mottled lean clay with and few fine gravels and lignites, very stiff	640	4/7/8	SS
		641	4/5/7	SS
		642	4/7/9	SS
4.8-9.0	--4.8-9.0 Light brown and gray mottled lean clay with scattered fine gravel and lignite, very stiff to hard, --gypsum crystals and sand seams @6.0	643	6/9/9	SS
		644	4/7/6	SS
		645	3/7/8	SS
9.0-10.8	--9.0-10.8 Brown lean clay with scattered fine gravel, stiff	646	5/10/15	SS
10.8-16.3	--10.8-16.3 Brown and gray sandy lean clay, very stiff with scattered fine gravel and lignite	647	4/9/15	SS
		648	4/8/11	SS
		649	5/10/13	SS
		650	5/9/15	SS
16.3-19.8	--16.3-19.8 Brown sandy lean clay, stiff to very stiff with scattered fine gravel, --gypsum crystals & sand seams @ 18.0	651	7/11/15	SS
		652	6/13/18	SS
19.8-21.0	--19.8-21.0 Light brown sandy lean clay with fine gravel and lignite, very stiff			
	Boring completed at depth of 21.0			

Columns also shown: Elevation (0 to -35), Profile, Moisture Content (%) (Including LL__PL) (10, 20, 30, 40), Undrained Strength (TSF) TV ● PP ■ (1, 2, 3, 4, 5), Strength Test, Recovery (ft).

SS--Split Spoon Sampler

SPT corrected N60 values not given -- just original blow sequence

University of Missouri - Columbia

Civil & EnvironmentalEngineering Department -Geotechnical Group

Project Name: US 54 - Fulton Site
Project NO: 906 RDT RI98-701
Location: 500 ft South of Richland Creek
Boring Number: MUC-12
Ground Elevation:
Logged By: Newton
Drilling Date: 10/11/2000
Driller: Barnett

Weather:
Auger Method: 4" Hollow Stem
Driller's Hole NO: 0-00-61
Conditions:
SPT Hammer Efficiency:
Drilling Equip: Versa Drill G7211

Elevation	Profile	Water Level	Depth	Soil Description	Sample NO	SPT N60	Moisture Content (%) (Including LL__PL)	Undrained Strength (TSF) TV ● PP ■	Sample Type	Strength Test	Recovery (ft)
0		GWT not encountered	0	--0.0-13.3 Tan and gray mottled sandy lean clay with scattered fine gravel, hard, lignite content increasing with depth, cobble from 8.6 to 8.8 ft. --gypsum crystals @11.0	653	7/12/9			SS		
					654	5/7/10			SS		
					655	6/9/12			SS		
-5			5		656	1/6/8			SS		
					657	5/10/15			SS		
					658	10/10/17			SS		
-10			10		659	3/8/11			SS		
					660	4/10/14			SS		
					661	6/11/13			SS		
				--13.3-21.0 Brown sandy, gravelly lean clay with few gray mottles and lignite, very stiff, gypsum crystals at 15.5 ft., scattered vertical seams of sand and crystals	662	6/11/12			SS		
-15			15		663	5/11/13			SS		
					664	6/13/18			SS		
					665	7/13/16			SS		
-20			20								
				Boring completed at depth of 21.0							
-25			25								
-30			30								
-35			35								

Moisture Content scale: 10 20 30 40
Undrained Strength scale: 1 2 3 4 5

SS--Split Spoon Sampler
SPT corrected N60 values not given, just original blow sequence